FLOOD MODELING AND MAPPING OF LOWER OMO GIBE RIVER BASIN

By

ASNAKE ESHETU GURMU

A Thesis Submitted to

The department of Civil Engineering for the Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

(Hydraulic Engineering)

ADDIS ABABA SCIENCE AND TECHNOLOGY UNIVERSITY

SEPTEMBER 2018

Declaration

I hereby declare that this thesis entitled **“Flood Modeling and Mapping of Lower Omo Gibe River Basin”** was composed by myself, with the guidance of my advisor, that the thesis contained herein is my own except where explicitly stated otherwise in the text, and that this work has not been submitted, in whole or in part, for any other degree or processional qualification. Parts of this work have been published in (Evaluation and Multi-Purpose Analysis of Omo-Gibe III Hydropower reservoir to Include Downstream Irrigation Development).

Name: ASNAKE ESHETU **Signature, Date:**

Approval Sheet I

This is to certify that the thesis prepared by **Mr. Asnake Eshetu Gurmu** entitled **"Flood Modeling and Mapping of Lower Omo Gibe River Basin"** and submitted in fulfillment of the requirements for the degree of Master of Science in Hydraulic Engineering complies with the regulations of the University and meets the accepted standards with respect to originality and quality.

Singed by Examining Board**:**

1.Brook Abate (phD)	____________________	____________________
(principal Advisor)	Signature	Date
2.Fitsum Tesfaye (phD)	____________________	____________________
(Internal Examiner)	Signature	Date
3.Tilahun Derib (phD)	____________________	____________________
(External Examiner)	Signature	Date
4.Melaku Sisay (phD)	____________________	____________________
(ERA PG, program Coordinator	Signature	Date
5.Mr.Seifu Sisay	____________________	____________________
Head, Civil Eng'g Department	Signature	Date
6. Brook Abate (phD)	____________________	____________________
(Dean,College of Architecture Civil Engineering)	Signature	Date

Abstract

Flood occurs repeatedly in Ethiopia and cause tremendous losses in terms of property and life, particularly in the lowland areas. The majority of flood disasters victims are poor people living in nearby stretch of floodplain. This research involves the integration of Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) and Hydrologic Engineering Center-River Analysis System (HEC-RAS) with Geographic Information Systems (GIS) to develop a regional model for floodplain determination and representation. Triangulated Irregular Network (TIN) was prepared from the digital elevation model (DEM) of the study area in GIS. Required data sets as stream centerline, banks, flow paths and cross sections were prepared in HEC-GeoRAS (Geospatial River Analysis system) thus, creating import file and imported in HEC-RAS. In HEC-RAS, boundary condition for downstream were defined. Similarly, flood-discharges for different return periods were also inputted and steady flow analysis was done for the results. For modeling, the daily discharge and rain-fall was used for HEC-HMS Calibration and Validation both the results showed good match between measured and simulated precipitation data with acceptable range, coefficient of determination(R^2) (0.68 to 0.88 for Calibration and 0.62 to 0.86 for Validation) and Nash-Sutcliffe efficiency(ENS) (0.5 to 0.88 for Calibration and 0.61 to 0.85 for Validation) and also, Ethiopian Road Authority (ERA) Intensity-frequency-duration curve were used for frequency storm analysis. The hydrologic frequency model was used for determining the peak flow discharge for return periods of 2, 10, 50 and 100 years and the result was found to be 484.4m^3/s,845.7m^3/s, 1222.1m^3/s and 1400.7m^3/s respectively. Additionally, the value derived by the hourly data of the HEC-HMS was compared with different frequency analysis methods. The study describes the flood extent and depth in the area for different flow conditions derived from the historical flow data of the Omo -Gibe River. According to the flood map, the flooded area for the return periods 2, 10, 50 and 100 years were 44.951 km^2, 74.986 km^2,76.4km^2 and 76.94km^2 respectively.

key words: Lower Omo-Gibe, Kuraz district, HEC-HMS, HEC-RAS, TIN, flood map, Gibe III Dam, Abelti gauging station, Stream flow.

Acknowledgements

First and for most, I would like to thank my holy father, God and his mother, for his care, support and endless love during my stay in Addis Ababa Science and Technology University. I am indebted to my respected Advisor, Dr. Brook Abate for his continuous guidance, advice and expedience from the proposal preparation to thesis finalization. His-constructive comments, untiring help, guidance and practical suggestions inspired me to- accomplish this work successfully. I would like to express my deepest gratitude to Dr. Adanech for her support in material and advise necessary for the preparation of the final thesis. I remember Mr. Nigusse for his friendly support in sharing knowledge which initiated me to continue the research in a safe way. His smiley and bright face make me see myself where I am. I express my special gratitude to Mr. Nigusse for his unrepayable idea, advice, suggestion and comments. My classmate friends, you are so friendly and you deserve my genuine appreciation. I enjoyed pleasant time with you. Thank you so much all of you. Finally, I offer my greatest appreciation to my Family for their Support me to do my thesis on time.

Table of Contents

List of Tables

List of Figures

List of Acronyms

Amsl	Above Mean Sea Level
CN	Curve Number
DEM	Digital Elevation Model
DTM	Digital Terrain Model
GeoHMS	Geospatial Hydrological Modeling
GOF	Goodness-of-Fit Tests
HEC-HMS	Hydraulic Engineering Center for Hydrologic Modeling System
HEC-RAS	Hydraulic Engineering Center for River Analysis System
IA	Initial Abstraction
LU	Land Use
OGMP	Omo Gibe Master Plan
MoWIE	Ministry of Water, Irrigation and Electricity
NMA	National Meteorological Agency
RF	Rain Fall
TIN	Triangular Irregular Network
UH	Unit Hydrograph
USACE	United States Army Corps of Engineers
WFP	World Food Program
IDF	Intensity Duration Frequency
PET	Potential Evapotranspiration
DMC	Double Mass Curve
ASTER	Advanced Spaceborne Thermal Emission and Reflection Radiometer images

CHAPTER ONE

1. INTRODUCTION

1.1. Background

Flood is defined as a great flow of water, especially, a body of water, rising, swelling and overflowing over land surface. Flood control implies all measures taken to reduce the detrimental effect of flood. Flooding over the world is one of the worst natural disasters, in terms of economic losses. Accurate and current floodplain maps can be the most valuable tools for avoiding severe social and economic losses from floods. This updated floodplain maps also improve public safety and property. Early identification of flood-prone properties during emergencies allows public safety organizations to establish warning and evacuation priorities. Flooding occurs due to high stages in the river, which can be caused by the following reasons that are: too high discharges, backing up of the water and increase in bed levels, human intervention in the highlands areas at an ever-increasing scale (Asadi, 2013).

The need to study the cause and effect of flooding has begun since flooding has become a problem to society when people and their valuables become affected. Historically many solutions have been proposed to mitigate the effects of flooding but knowledge on the actual cause effect relation is lacking. With the advent of digital computers, much emphasis has been on simulating and modeling of flood events and related characteristics and such is the main concern of this paper. The challenge here is to develop a reliable flood model to simulate flood events for Lower Omo catchment (Ohimain, 2014).

Hydrologic models are used to estimate the peak discharge and volume of runoff/runoff hydrographs. Establishing a rainfall-runoff relationship was the central focus of hydrologic modeling from its simple form of unit hydrograph to rather complex models based on fully dynamic flow equations. As the computing capabilities are increasing, the use of these models to simulate a catchment has become a standard. Models are generally used as utility in various areas of water resources, in studying the impacts of human interference in an area such as land use change, deforestation and other hydraulic structures such as dams and reservoirs (Arekhi, 2012)

These hydrological Models with different approaches are used nowadays for different water resource development works. The choice of methods of estimation of peak discharge clearly depends on the data requirements and data availability. Among the different approaches, GIS based hydrological model systems are increasingly becoming major hydrological modeling tools because of its capability to handle the spatial variation of hydrological and physiographic inputs of the watershed. Several models, which either are embedded in the GIS (Geographical Information System) environment or have capability of importing the GIS derived spatial and temporal attributes, have been developed. One of which is the United States Army Corps of Engineers' HEC-HMS (the Hydrologic Engineering Center's- Hydrological Modeling System). The program simulates precipitation-runoff and routing processes, both natural and controlled (USACE, 2000).

Causes of flooding may be either natural or human induced. Natural causes may be high and long-lasting precipitation or extreme events such as earth quick and tsunamis. Man induced causes include failure of dam or levee. Mitigating in flood effects requires information on the flooding characteristics and how such characteristics propagate. Such information can be obtained through hydraulic models that are able to simulate flood events, depths, levels, velocity and timing over a distributed model domain and over the time dimension. Hydraulics models have the ability to solve such problems. HEC-RAS/HEC-GeoRAS is among the powerful model and is able to model flooding characteristics given that sufficient input data of good quality is available (Ohimain, 2014).

Open water flood forecasts are typically based on a two-step process in which a hydrological model was first used to route the flood and determine expected flow hydrographs at the site of interest. The peak flow from this flood routing analysis is then typically input into a steady flow hydraulic model, such as the HEC-RAS model, to determine the corresponding flood levels that would be expected along river reaches extending through populated areas along the flood plain (Ohimain, 2014). In addition to this, surface water profile of one of the channels where there is overtopping is computed using the HEC-RAS Hydraulic Model by employing the flood discharges calculated from HEC-HMS model in order to recommend mitigation measures for flood risk areas (Diabi, 2016).

In hydraulic flood modeling, availability of data in the required spatial and temporal resolution is vital. Topographic data is one of such data used as input in hydraulic flood modeling. Digital

Elevation Model (DEM) and/or its derivative Triangular Irregular Network (TIN) was the major source of topographic data for representing floodplain and river topography. However, availability and quality of the geometric data of river cross sections was a major limitation. For such availability of basic data sources, a high-resolution DEM is a prerequisite. Even though there is still a gap to find this high-resolution DEM to represent the topography of the study area, the 30m DEM is selected to best represent (Ohimain, 2014).

1.2. Problem statement

Floods will continue to cause serious economic and environmental losses. It is reported that flood disasters account for about one third of all-natural disasters in terms of their number and the economic losses. Flood and droughts are the world's costliest natural disasters, causing an average $6–$8 billion in global damages annually and collectively affecting more people than any other form of natural disaster (Lampros, 2009). An example is the flood that had been caused in 2006 in the Omo basin which inundated the Dasenech and Nyangatom Weredas killing 364 people and displacing between 6000 and 10000 in Kuraz District, South Omo.Nearly 3000 livestock are also reported to have perished due to the flooding (OCHA UN office of the co-ordination of humanitarian affairs, 2006). The majority of the source of the flood is from the Ethiopian highlands where the Lake is sustained by the inflows of Ethiopia's Omo River, which alone provides 90 % of the lake inflow (Avery, 2010).With the upstream interventions of the Gibe I, II, III and IV already built and under construction; the downstream flood-based farming could be affected by flood.

The lower Omo plain is one of the flood-affected areas in Ethiopia. It regularly inundates the settlements in the downstream part of the basin and affects the community benefits from flood and recession agriculture. Necessity to conduct this research was frequent happening of floods in the area. As the region is known as crop productive area of the country, most of the crops may be suffered. Information regarding the flooding characteristics and its effect are essential for flood management bodies for decision making in flood management strategies such as construction of flood protection structures (engineering structures), to develop flood emergency plan and human settlement planning. With the advancement of new technology on flood modeling such as the hydraulic modeling (HEC-RAS) and GIS these days, it is possible to model flood extent, depth, distribution etc. in the temporal and spatial dimensions. Past flood study in the area however ignores these applications.

1.3. Objectives

1.3.1. General Objective

The general objective of this research is to prepare flood modeling and mapping of Lower Omo Gibe River Basin Catchment using Rainfall-Runoff model HEC-HMS (Hydrologic Engineering Center-Hydrologic Modeling System) and Hydraulic modeling.

1.3.2. Specific objectives

The specific objective of the thesis is: -

1. To fit the best flood probability distribution for the Easy fit.
2. To estimate peak flood discharge for different design precipitations using HEC HMS.
3. To develop flood map for different design periods for the River and flood plain using HEC-RAS.

1.4. Research Questions

1. What is the best fit probability distribution of what for Omo Gibe area?
2. How much are peak flood discharge for different design precipitations?
3. How much is the spatial flood incidence distribution using HEC-RAS?

1.5. Significance of study

The flooding characteristics and its effect are essential for flood management bodies for decision making in flood management strategies. The study describes the flood extent and depth in the area for different flow conditions derived from the historical flow data of the Omo Gibe River. Estimation of hydrologic design parameters (peak discharge and runoff volume/runoff hydrographs) using a GIS based semi distributed hydrological model (HEC-HMS). The finding of this study was redounding to the benefit of the society considering that advancement of new technology on flood modeling such as the hydraulic modeling (HEC- RAS) and GIS these days, it is possible to model flood extent, depth, distribution.

CHAPTER TWO

2. LITERATURE REVIEW

2.1. Fitting a flood Probability Distribution

A probability density function (PDF) is a continuous mathematical expression that determines the probability of a particular event. If a prediction is to be based on a set of hydrologic data, then the distribution that best fits the set of data may be expected to give the best estimates usually an extrapolation of the probability of an event occurring (Research, 2013).The Six probability distributions for Hydrologic Variables selected for this study are General Extreme value, Log-Pearson type III, Normal, Extreme value type 1(EV-I), Lognormal. Their essential properties are given in Table 2.1.

Table 2-1 Probability distribution parameters in relation to sample moments (Alem, 2018)

Distribution	Probability density function	Range
Noomal	$f(x)=\frac{1}{\sqrt{2\pi\sigma^2}}e^{-(x-\mu)^2/2\sigma^2}$	$-\infty \leq x \leq \infty$
Lognormal	$f(x)=\frac{1}{x\sqrt{2\pi\sigma^2}}\exp\{-(\ln x-\mu)^2/2\sigma^2\}$	$-0 \leq x \leq \infty$ $-\infty < \mu < \infty, \sigma > 0$
Gamma	$f(x)=\frac{1}{\beta^{\alpha}\Gamma\alpha}x^{\alpha-1}e^{-x/\beta}$	$0 \leq x \leq \infty$ $\alpha \geq 0, \beta \geq 0$
Weibull	$f(x)=\frac{\alpha}{\beta}\left(\frac{x}{\beta}\right)^{\alpha-1}\exp\left\{-\left(\frac{x}{\beta}\right)^{\alpha}\right\}$	$x \geq 0$
Inverse Gaussain	$f(x)=\left[\frac{\lambda}{2\pi x^3}\right]^{1/2}\exp\frac{-\lambda(x-\mu)^2}{2\mu^2 x}$	$x > 0, \mu > 0$ and $\lambda > 0$
Generalized Extreme value	$f(x)=\frac{1}{\sigma}\left[1-k(\frac{x-\mu}{\sigma})\right]^{1/k-1}\exp\left[-\left\{1-k(\frac{x-\mu}{\sigma})\right\}\right]^{1/k}$	$-\infty \leq x \leq \infty$ $k \neq 0$

Where: f(x) = Probability density function

σ^2=Variance

μ= mean

The main limitations of the normal distribution for describing hydrologic variables are that it varies over a continuous range [-∞, ∞], while most hydrologic variables are non-negative, and that it is symmetric about the mean, while hydrologic data tend to be skewed and also the mean, Standard deviation are the parameter for the normal distribution. Gamma distribution: is useful for describing skewed hydrologic variables without the need for log transformation. The two-parameter gamma distribution (parameters β and α) has a lower bound at zero, which is a disadvantage for application to hydrologic variables that have a lower bound larger than zero and also λ and μ are the parameter for the Inverse Gaussian probability distribution.

2.1.1. Extreme value Type I (EV-I) distribution

The Extreme Value Type 1 (Gumbel) distribution, one of the most commonly used distribution in flood frequency analysis. The distribution is based on theory of extremes and it is considered appropriate for this analysis as annual series data used for this study is composed of peak values (extreme values) for each year. The PDF and other parameters relating to the distribution are given in Table 2.1. According to (Ven Te Chow, 1998).there are a number of distributions in hydrology used to analyze the probability of occurrence of a stream flow. For extreme value type one distribution, (Ven Te Chow, 1998) derived the expression.

$$K_T = -\frac{\sqrt{6}}{\pi}\{0.5772+\ln[\ln(\frac{T}{T-1})]\} \quad (2.1)$$

$$X_T=\mu + K_T\,\sigma \quad (2.2)$$

Where: K_T=frequency factor and

σ = Standard deviation

2.1.2. Normal and Lognormal Probability distributions

The Normal distribution is the most familiar probability distribution (Alem, 2018), Its PDF is given by:

$$f(x)= \frac{1}{\sigma\sqrt{2\pi}}*\exp(-\frac{(x-\overline{x})^2}{2\sigma^2}) \quad (2.3)$$

It is defined by two distribution parameters; the mean ($\bar{x}$), and standard deviation (σ)evaluated by:

$$\bar{x} = \frac{1}{N}\sum_{i=1}^{N} x_i \ldots\ldots (2.4)$$

Where: x_i is the magnitude of the i^{th} event

N is the total number of events

The standard deviation (σ) which is a measure of the dispersion or spread of data set is given by:

$$\sigma = \frac{1}{N} * \sum_{i=1}^{N}(Xi - \mu)^2) \ldots\ldots (2.5)$$

The normal distribution describes many random processes but it generally does not provide satisfactory fit for flood discharge and other hydrologic data (Ajumuka, 2013)

A particular event x can be related to the probability of exceedance P by the following equation:

$$X = \bar{x} + K * \sigma \ldots\ldots (2.6)$$

where k is the frequency factor. Though, the normal distribution is not well suited to hydrologic data, the related distribution; the lognormal distribution works reasonably well (Research, 2013)

The Log- normal distribution assumes that the logarithms of the discharge are themselves normally distributed. The equation describing normal distribution is modified for use in the case of log normal distribution if the following substitution is made with: -

$$y_i = \log x_i \ldots\ldots (2.7)$$

With x replaced by y, the mean of the logarithms and standard deviation becomes

$$\bar{x} = \frac{1}{N}\sum_{i=1}^{N} \log x_i \ldots\ldots (2.8)$$

$$\sigma = \frac{1}{N} * \sum_{i=1}^{N}(\log x_i - \mu)^2) \ldots\ldots (2.9)$$

2.1.3. Log-Pearson type III

The problem with most hydrologic data was that an equal spread does not occur above and below the mean. The lower side is limited to the range from mean to zero while there is theoretically no limitation on the upper range thereby contributing to what is called a skewed distribution. The coefficient of skew (G) is defined mathematically by:

$$G = \frac{N * \sum(X - \overline{X})^3}{(N-1)(N-2) * S^3} \ldots\ldots (2.10)$$

In wich: X=Logarithm of annual peak flow

N=number of items in data set

$\bar{x}$ = mean logarithm

S=Standard deviation of logarithms

G=Skew coefficient of Logarithms

It is to take account of the skew that may exist in data that the log Pearson type III distribution was developed to improve the fit (Ajumuka, 2013). The distribution uses three parameters namely: mean, standard deviation and skew coefficient which are obtained using equations (2.8), (2.9) and (2.10) respectively.

2.2. Easy Fit Software

Easy Fit is a data analyzer and simulation software which allows to fit probabilistic distributions to given data samples, simulate them, choose the best fitting sample, and implement the results of analysis to take better decisions. This software can be used as a Windows compatible program, and also as an add-on to Excel spread sheets (Pakgohar, 2014).

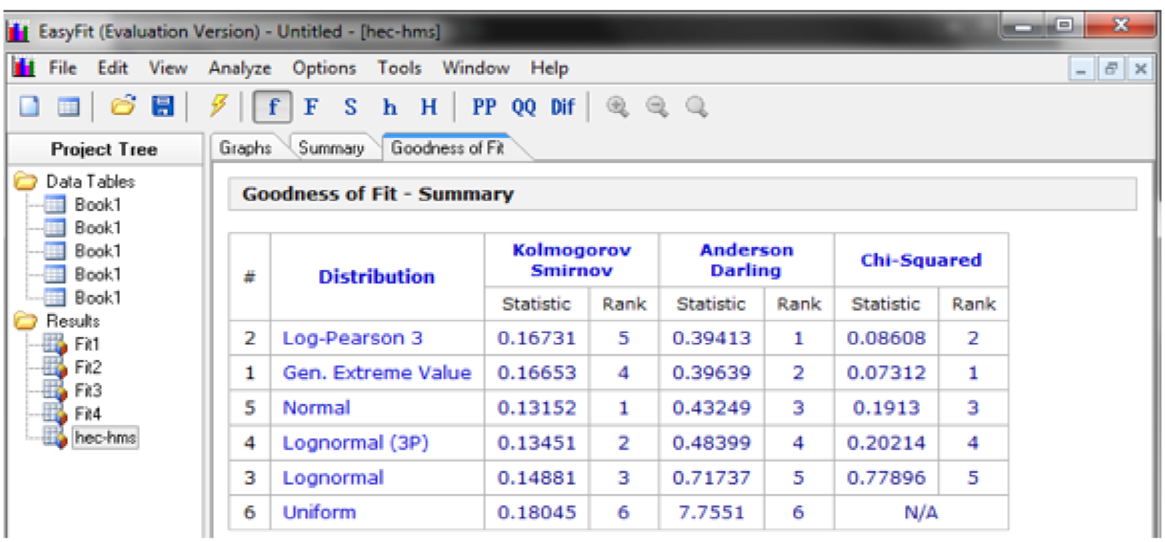


Figure 2-1 Easy fit software Evaluation for the Goodness of fit summary

When a certain distribution is chosen as data distribution, it is expected to fit suitably with data, so we are ready to practically fit those data with the distributions. Data fitting process includes using certain statistical techniques which allow estimate fitness parameters based on sample data.

The distribution fitting software can be very useful in this sense. Clearly this program was using the methods of parameter estimation on the best-known distributions, though saving your time and letting to concentrate on data analysis task. Since researchers sometimes want to fit several different distributions to the data simultaneously, they need to estimate parameters of each distribution separately (Mehrannia, 2014). Input to the data fitting software usually the data in any accepted format and distributions which want to fit with them. Fitting choices Output results of the fit would include graphs related to raw input data, distribution or improved fitting parameters, graphs of fitted distribution, additional graphs and tables which help to choosing best fit for the data.

2.2.1. Statistical Theories

In this section we try to present a general review of statistical theories and techniques in data fitting and Goodness-of-Fit test field. We would have an overview on statistics and related graphs as well shown on Appendix H.

2.2.2. Goodness-of-Fit Tests (GOF)

The goodness of fit (GOF) tests measure the compatibility of a random sample with a theoretical probability distribution function. In other words, these tests show how well the distribution selected fits to the data. The results are presented in the form of interactive tables that help to decide which model describes your data in the best way. A couple of goodness-of-fit test have been conducted such as Kolmogorov-Smirnov test, Anderson-Darling test along with the chi-square test at significance level (α=0.05) for choosing the best probability distribution (Alem, 2018).

2.2.2.1. Kolmogorov-Smirnov Test

This test was used to decide if a sample comes from a hypothesized continuous distribution. It is based on the empirical cumulative distribution function (ECDF). Assume that to have a random sample $X_1, ..., X_n$ from some distribution with CDF F(x). The empirical CDF is denoted by:

$$F_n(x)=\frac{1}{n}*[\text{Number of observations} \leq x] \quad \ldots\ldots (2.11)$$

The Kolmogorov-Smirnov statistic (D) is based on the largest vertical difference between the theoretical and the empirical cumulative distribution function:

$$D = \max(F(xi) - \frac{i-1}{n}, \frac{i}{n} - F(xi) \quad \ldots\ldots (2.12)$$

2.2.2.2. Anderson-Darling Test

The Anderson-Darling procedure is a general test to compare the fit of an observed cumulative distribution function to an expected cumulative distribution function. This test gives more weight to the tails than the Kolmogorov-Smirnov test. The Anderson-Darling statistic (A^2) is defined as

$$A^2 = -n - \frac{1}{n}\sum_{i=1}^{n}(2i-1)*[\ln F(X_i) + \ln(1-F(X_{n-i+1}))] \quad \ldots\ldots\ldots\ldots (2.13)$$

2.2.2.3. Chi-Squared Test

The Chi-square test assumes that the number of observations is large enough so that the chi-square distribution provides a good approximation as the distribution of test statistic. The Chi-squared statistic is defined as.

$$x^2 = \sum_{i=1}^{k}\frac{(Oi-Ei)^2}{Ei} \quad \ldots\ldots\ldots\ldots (2.14)$$

Where: Oi =is the observed frequency

E_i =Expected frequency $E_i = F(X_2) - F(X_1)$

i= number of observations (1,2, 3……. k)

F = the CDF of the probability distribution being tested.

The observed number of observation (k) in interval 'i' is computed from equation given below

$$K = 1 + \log_2^n \quad \ldots\ldots\ldots\ldots (2.15)$$

n = sample size

This equation is for continuous sample data only and is used to determine if a sample comes from a population with a specific distribution (Olofintoye, 2009).

2.3. Rainfall Runoff Modeling

The use of rainfall-runoff models inevitably extends the lead time. These models relate current rainfall in a catchment to river flows or reservoir inflows. For the forecasting chain starting from the measured rainfall and using it as input to a rainfall-runoff model, forecasting lead time becomes a function of delays, mostly due to the filling of the soil storage and surface waters travel time. Rainfall-runoff models may be either lumped (i.e. using a single rainfall input spatially averaged across the catchment) or distributed (i.e. accounting to some extent for the distribution of rainfall).

River flows may be forecasted at specific points along a river to provide warnings at these points or used as input to flood routing models to provide warnings further downstream. The following are some of the rainfall- runoff models (Asadi, 2013). Rational method, SCS and unit hydrograph method, analysis of stream gage data, suitable computer programs and Hydrologic modeling HEC-HMS.

2.3.1. Hydrologic model (HEC-HMS)

HEC-HMS (the Hydrologic Engineering Center's Hydrological Modeling System) Model developed by the US Army Corps of Engineers is designed to simulate the precipitation-runoff processes of dendritic watershed systems (USACE, 2005). HEC-HMS conceptually represents watershed behavior as different components of runoff processes. It has an appropriate representation of the hydrological system, and its specification depends upon the information needs of the hydrological study. Hydraulic modeling and inundation mapping, the main objective is to accurately predict catchment outflows from upstream sub catchments and flood wave propagation along the drainage network. A model relates something unknown (the output) to something known (the input). In the case of models included in HEC-HMS, the known input is precipitation and the unknown output is runoff, or the known input is the upstream flow and the unknown output is the downstream flow (USACE, 2005).

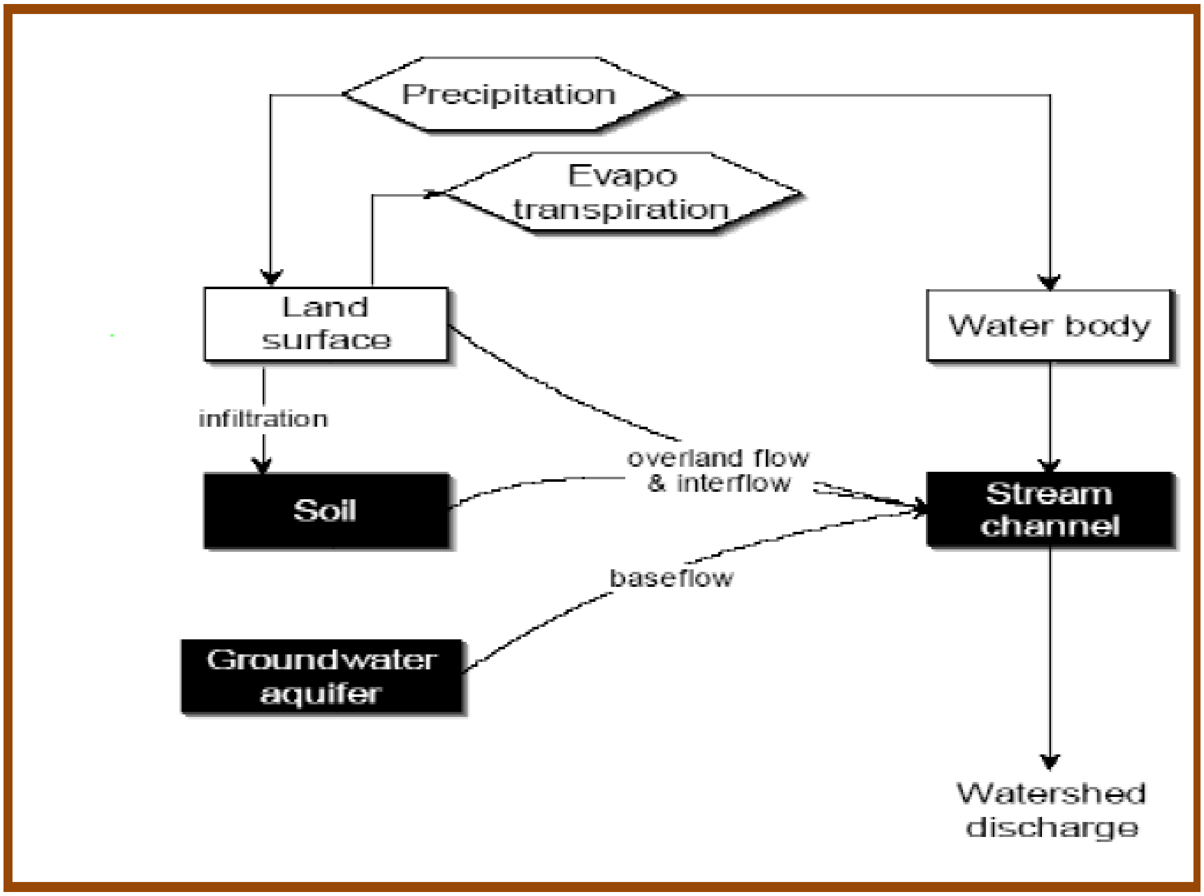


Figure 2-2 Typical HEC-HMS representation of watershed runoff (USACE, 2000).

2.3.2. The Analytical Components of HEC-HMS

HEC-HMS consists of separate models of the major hydrological processes and transports. It consists of runoff volume models, models of direct runoff (overland flow and interflow), base flow models, channel flow models. HEC-HMS gives flexibility to the user by providing each component with suit of models. The user was chosen a suitable combination of models depending on the availability of data, the purpose of modeling and the required spatial and temporal scale (Asadi, 2013).

2.3.2.1. Runoff-Volume Models

As illustrated by Figure 2.2, HEC-HMS computes runoff volume by computing the volume of water that is intercepted, infiltrated, stored, evaporated, or transpired and subtracting it from the precipitation. Interception, infiltration, storage, evaporation, and transpiration collectively are referred to in the HEC-HMS program and documentation as losses (Feldman, 2000).HEC-HMS considers that all land and water in a watershed can be categorized as either directly-connected impervious surface, or pervious surface. Directly connected impervious surface in a watershed is that portion of the watershed for which all contributing precipitation runs off, with no infiltration, evaporation, or other volume losses. Precipitation on the pervious surfaces is subject to losses. The following alternative models are included to account for the cumulative Losses. However, only some of the applicable methods in the context of this research paper are described below (Feldman, 2000).

The Initial and Constant Rate Loss Model

The Deficit and constant rate model

The SCS Curve Number (CN) Loss Model

The Green and Ampt Loss Model

2.3.2.2. SCS Curve Number Loss Model

The Soil Conservation Service (SCS) Curve Number (CN) model estimates precipitation excess as a function of cumulative precipitation, soil cover, land use, and antecedent moisture, using the following equation:

$$P_e = \frac{(P-I_a)^2}{P-I_{a+S}} \quad \text{(2-16)}$$

Where:

P_e = accumulated precipitation excess at time t.

P = accumulated rainfall depth at time t.

I_a= the initial abstraction (initial loss) and

S = potential maximum retention, a measure of the ability of a watershed to abstract and retain storm precipitation.

Until the accumulated rainfall exceeds the initial abstraction, the precipitation excess, and hence the runoff, will be zero. From analysis of results from many small experimental watersheds, the SCS developed an empirical relationship of I_a and S:

$$I_a=0.2*S \quad \text{(2-17)}$$

Therefore, the cumulative excess at time t is:

$$P_e=\frac{(P-I_a)^2}{P-I_{a+S}} \quad \text{(2.18)}$$

Incremental excess for a time interval was computed as the difference between the accumulated excess at the end of and beginning of the period.

The maximum retention, S, and watershed characteristics are related through an intermediate parameter, the curve number (commonly abbreviated CN) as:

$$S = \frac{25400-254*CN}{CN} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.19)$$

CN values range from 100 (for water bodies) to approximately 30 for permeable soils with high infiltration rates (Arekhi, 2012).

Estimating CN

The **CN (**land use, practice, soil permeability, soil hydrologic group) for a watershed can be estimated as a function of land use, soil type -using tables published by the SCS. For convenience, **Appendix A** of this document includes Soil types developed by the SCS. For a watershed that consists of several soil types and land uses, a composite CN is calculated as:

$$CN_{\mathrm{Composite}} = \frac{\sum A_{i}\, CN_{i}}{\sum A_{i}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.20)$$

Where: - $CN_{\mathrm{Composite}}$= the composite CN used for runoff volume computations; i = an index of watersheds sub-divisions of uniform land use and soil type; CN_i = the CN for subdivision i; and A_i= the drainage area of subdivision i.

$CN_{\mathrm{Composite}}$= (10370*73.34+6109*70.47+2416.53*72.98)/18894 =72.37.

The CN shown are composite values for directly connected impervious area and open space. If CN for these land uses are selected, no further accounting of directly connected impervious area is required (Asadi, 2013).

Hydrological Soil Groups for Ethiopia

Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Permeability and infiltration are the principal data required to classify soils into Hydrologic Soils Groups (HSG). Based on infiltration rates, the Soil-Conservation Service (SCS) has divided soils into four hydrologic soil groups as follows:

Group A: Sand, loamy sand or sandy loam are soils having a low runoff potential due to high infiltration rates. These soils primarily consist of deep, well-drained sands and gravels.

Group B: Silt loam, or loam are soils having a moderately low runoff potential due to moderate infiltration rates. These soils primarily consist of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C: Sandy clay loam are soils having a moderately high runoff potential due to slow infiltration rates. These soils primarily consist of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D: Clay loam, silty clay loam, sandy clay, silty clay or clay are soils having a high runoff potential due to very slow infiltration rates. These soils primarily consist of clays with high swelling potential, soils with permanently-high water tables, soils with a clay-pan or clay layer at or near the surface, and shallow soils over nearly impervious parent material. Data from direct field measurements on soil permeability and infiltration rates for Ethiopian soils are very limited. Data is generally available only for soil types located near major irrigation projects and agricultural research stations. The hydrological soils groups presented in Appendix A are based on limited field measurements and from profile morphology and physical characteristics and are subject to further review and refinement (ERA, 2013).

2.3.2.3. Direct-Runoff Models

Modeling direct runoff is transformation of the excess precipitation into point runoff at a given point outlet. HEC-HMS includes two options, systems type and conceptual type of transformation. The systems type transformation included in HMS consists of Snyder's unit hydrographs model, SCS UH model, Clark's model, Modified Clark's model. The conceptual model includes only a kinematics wave model of overland flow (Arekhi, 2012).

SCS Unit Hydrograph model

The Soil Conservation Service (SCS) proposed a parametric UH model; this model is included in HEC-HMS. The model is based upon averages of UH derived from gauged rainfall and runoff for a large number of small agricultural watersheds throughout the US.

Basic Concepts and Equations:

At the heart of the SCS UH model is a dimensionless, single-peaked UH. This dimensionless UH, expresses the UH discharge, Ut, as a ratio to the UH peak discharge, Up, for any time t, a fraction of Tp, the time to UH peak.

Research by the SCS suggests that the UH peak and time of UH peak are related by:

$$U_P = C * \frac{A}{T_P} \quad (2.21)$$

in which A = watershed area; and C = conversion constant (2.08 in SI system). The time of peak (also known as the time of rise) was related to the duration of the unit of excess precipitation as:

$$T_P = \frac{\Delta t}{2} + t_{lag} \quad \text{.....} \quad (2.22)$$

in which Δt = the excess precipitation duration (which is also the computational interval in HEC-HMS); and t_{lag} = the basin lag, defined as the time difference between the center of mass of rainfall excess and the peak of the UH. [Note that for adequate definition of the ordinates on the rising limb of the SCS UH, a computational interval, Δt, that is less than 29% of t_{lag}must be used (USACE, 2005). When the lag time is specified, HEC-HMS solves Equation (2.18) to find the time of UH peak, and Equation (2.21) to find the UH peak. With U_P and T_P known, the UH can be found from the dimensionless form, which is included in HEC-HMS, by multiplication.

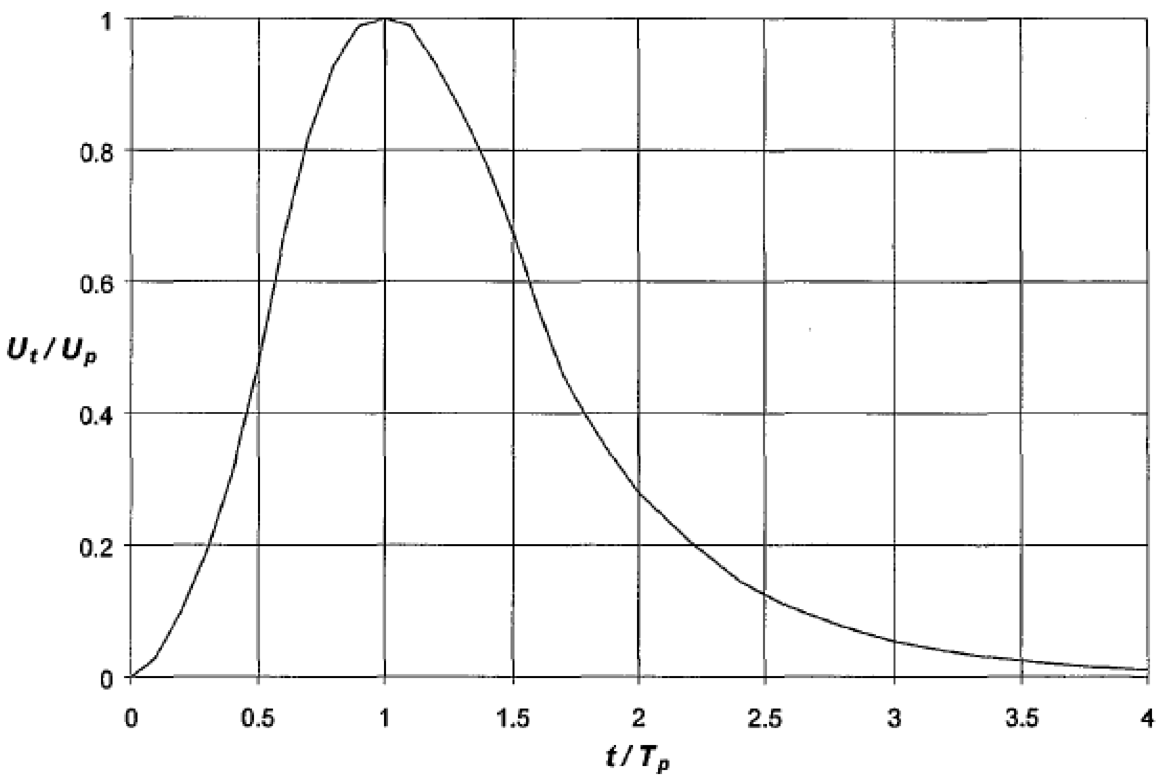


Figure 2-3 SCS Unit hydrograph. Source: HEC-HMS Technical Reference Manual,2000

2.4. HEC-GeoHMS

HEC-GeoHMS (Hydrologic Engineering Center's Geospatial Hydrologic Modeling System) has been developed as a geospatial hydrology tool kit for engineers and hydrologist. The program is an extension of Arc GIS and allows users to visualize spatial information, document watershed characteristics, perform spatial analysis, delineate sub-basins and streams, construct inputs to hydrologic models, and assist with report preparation. Eight data sets can be derived from DEM that collectively describe the drainage patterns of the watershed (Davis, 2009). HEC-GeoHMS provides the connection for translating GIS spatial information into hydrologic models. The end result of the GIS processing is a spatial hydrology database that consists of the digital elevation model (DEM), soil types, land use information, rainfall, etc. HEC-GeoHMS operates on the DEM to derive sub-basin delineation and to prepare a number of hydrologic inputs. 30m resolutions DEM were used for this case. HEC-HMS accepts the hydrologic inputs as a starting point for hydrologic modelling (Asadi, 2013).The relation between GIS, HEC-GeoHMS, and HEC-HMS is illustrated in Figure 2.4.

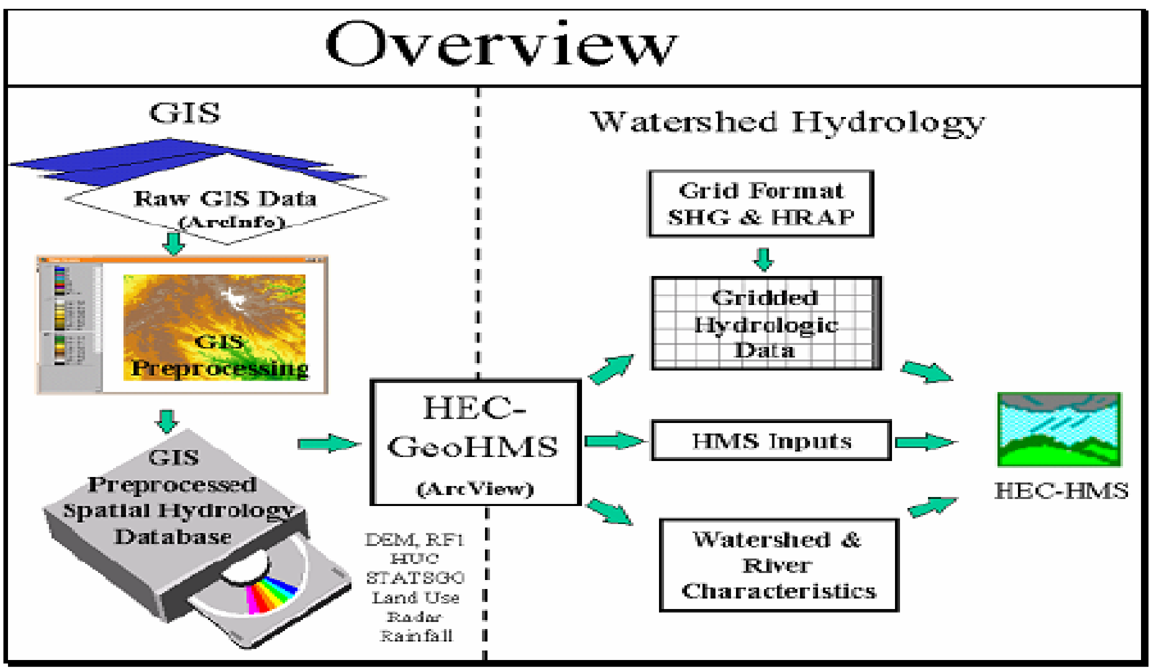


Figure 2-4 Overview of GIS, HEC-GeoHMS and HEC-HMS. Source: (Davis, 2009)

The following procedures describe the major steps in starting a project research and taking it through the GeoHMS development of a hydrologic model using DEM. These are:

i. Terrain Model Pre-Processing: Data-management, Dem-reconditioning, build walls, fill sinks, Flow-Direction, Flow-accumulation, stream definition, stream segmentation, Catchment Grid delineation etc.

ii. Hydrologic Processing

Basin Processing

Stream and Watershed Characteristics

HMS Model Files

iii. Hydrologic Parameters and HEC-HMS

2.4.1. Terrain Model Pre-Processing

The steps consist of computing the fill sinks, flow direction, flow accumulation, stream definition, stream segmentation, watershed delineation, watershed polygon processing, stream segmentation and watershed aggregation. These steps can be done step by step or in a batch manner. Watershed and stream delineation developed in this step is preliminary and they are used in later steps for sub-basin and stream delineation. Terrain pre-processing was performed in the Main View document of ArcGIS version 10.3 (Davis, 2009).

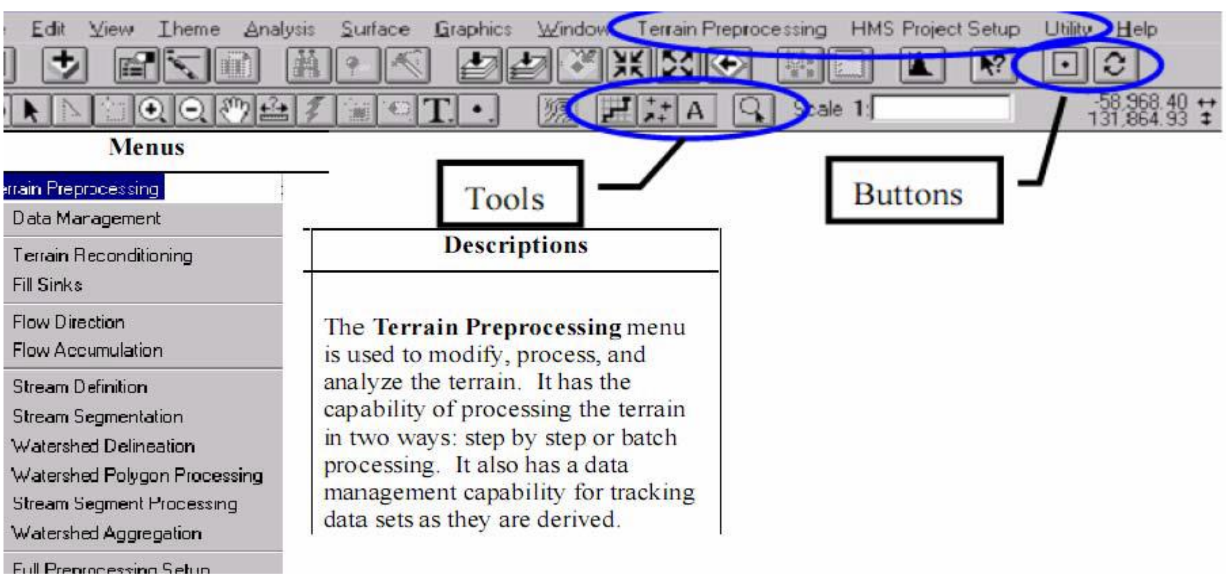


Figure 2-5 Over-view of Tools, Buttons in Terrain preprocessing. Source: ArcGIS version 10.3

2.4.2. Hydrologic Processing

The steps consist of computing basin processing, stream and watershed characteristics and HMS Model Files, was performed in the Project View document of Arc GIS GUI (Graphical user interface).

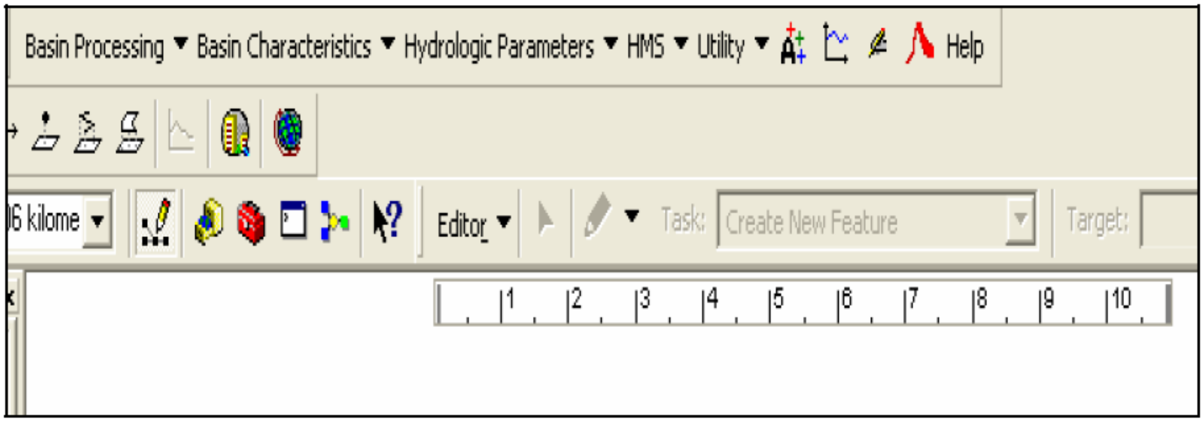


Figure 2-6 Over view of hydrologic processing tools (HEC-GeoHMS user's manual 2009).

2.4.3. Channel Flow

Also known as routing models. The routing models available in HEC-HMS includes: Lag, Muskingum, Modified puls, also known as storage routing, Kinematic-Wave and Muskingum-cunge. Each of these models computes a downstream Hydrograph, given an upstream hydrograph as boundary condition. Only Muskingum method was used for this study and is discussed below**.**

The hydrographs from the upper basins would be combined with the lower basin hydrograph at the watershed outlet. Routing parameters should be determined to compute lag and attenuation on the upper basin hydrographs before adding them to the lower hydrograph. The parameters, which describe the reach, determine the relationship between the upstream and downstream hydrograph. Assuming an incompressible liquid, the equation of conservation of mass for a channel reach can be written as:

$$\text{I-O} = \frac{d_s}{d_t} \quad \ldots\ldots \quad (2.23)$$

Where: - I-inflow

O-outflow

S-Storage

t-Time

Hydrologic reach routing methods are based on solving the above equation without explicitly accounting for momentum or energy considerations (Feldman, 2000).

Muskingum Model

The Muskingum method is often used in channel routing. The method was dependent primarily upon the following factors: the number of integer steps for the routing, Muskingum K coefficient in hours and Muskingum X coefficient. This model uses a simple finite difference approximation of the storage continuity equation. Storage is modeled as the sum of prism storage and wedge storage. The Muskingum defines the storage as:

$$S_t=KO_t+KX(I_t-O_t)=K(XI_t+(1-X)O_t \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(2.24)}$$

Where K=travel time of the flood wave through routing reach; and X=dimensionless weight ($0 \leq X \leq 0.5$). The quantity $K(XI_t+(1-X)O_t$ is a weighted discharge.

If the storage in the channel is controlled by downstream conditions, such that storage and outflow are highly correlated, then X=0. In this case equation 2.18 resolves to S=KO; this is the linear reservoir model. If X=0.5, equal weight is given to inflow and outflow, and the result is a uniformly progressive wave that does not attenuate as it moves through the reach.

Estimating the model parameter

Constraints on the parameters.as noted, the feasible range for the parameter X is (0,0.5). Calibrating the model using observed flows. If observed inflow and outflow hydrographs are available, the Muskingum model parameter K can be estimated as the interval between similar points on the inflow and outflow hydrographs. For example, K can be estimated as the elapsed time between the centroid of areas of the two hydrographs, as the time between the hydrograph peaks, or as the time between midpoints of the rising limbs. Once K is estimated, X can be estimated by trial and error. But in calibration both K and X, may be estimated by trial and error.

Experience has shown that for channels with mild slopes and over bank flow, the parameter X were approach 0.0.for steeper streams, with well-defined channels that do not have flows going out of bank, X were closer to 0.5. Most natural channels lie somewhere in between these two limits, leaving room for engineering judgment (Arekhi, 2012).

2.5. Hydraulic Modeling

Hydraulic models, in general, are more physically based on the hydrologic models since the only have one parameter (the roughness coefficient) to calibrate. The full unsteady flow equation has the capability to simulate the widest range of flow situations and channel characteristic. The basic data requirement for hydraulic routing techniques include: flow data, channel geometry, roughness coefficients, and internal boundary conditions. Hydraulic modeling was further subdivided into steady flow analysis and unsteady flow analysis. In unsteady flow, time dependent changes flow rate is analyzed explicitly as a variable, while steady flow analysis models neglect time all together (US Army Corps of Engineers, 2010).

2.5.1. Hydro-dynamic model: (HEC-RAS)

HEC-RAS, developed by the United States Army Corps of Engineers Hydrologic Engineering Center-River Analysis System, was intended for performing one-dimensional hydraulic calculations for a full network of natural and constructed channels. The system can calculate water surface profiles for both steady and unsteady gradually varied flow. The steady flow system is designed for application in floodplain management studies. Also, capabilities are available for assessing the change in water surface profiles due to channel improvements, and levees. HEC-GeoRAS, an ArcView GIS extension, creates a HEC-RAS import file containing geometric attribute data from a digital terrain model (DTM) and performs post processing of results exported from HEC-RAS. Main parameters needed are cross-sections for river and flood plain including left and right bank locations and flow paths, roughness coefficients (Manning's n), and contraction and expansion coefficients (Tanka, 2004) .

2.5.1.1.Governing Equation of the model

Energy Equation

For open channel flow, the total energy per unit weight (energy head) has components of elevation head, pressure head, and velocity head (Figure 2.7):

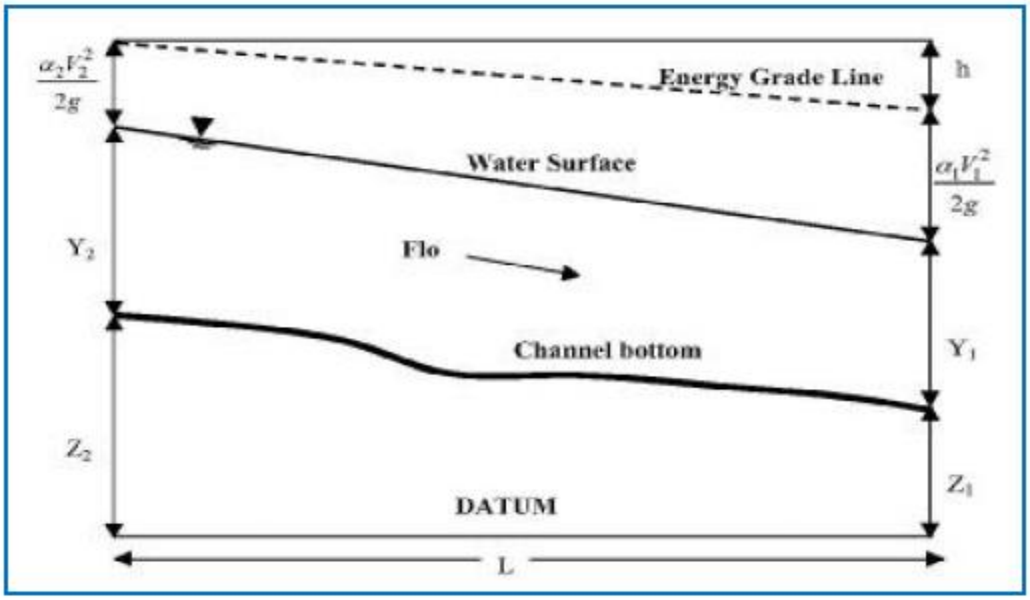


Figure 2-7 Energy equation parameters for gradually varied flow. Source: (US Army Corps of Engineers, 2010)

$$H = Y + Z + \frac{\alpha V^2}{2g} \quad \text{...} \quad (2.25)$$

Where: H = energy head (m)

Z= base channel elevation (m)

V= velocity weighting coefficient

Based on these parameters, the water surface elevation were the sum of y and z. The change in energy head between adjacent cross sections is equal to the head loss:

$$H_1 = h_2 + h_L \text{...} \quad (2.26)$$

Where: H_1= energy head at cross section 1 (m)

h_2= energy head at cross section 2 (m)

h_L= energy head loss (m)

The head loss between the two cross sections were the sum of friction head loss and flow contraction/expansion head loss. Friction losses result from shear stress between the water and channel bottom

$$h_f = LS_f \text{...} \quad (2.27)$$

Where: h_f= friction head loss (m)

L = distance between adjacent cross sections (m)

Contraction/expansion head losses can occur as a result of the formation of eddies wherever there is a contraction or expansion of the channel.

$$h_o = C\left[\left(\frac{\alpha_2 V_2^{\,2}}{2g}\right)-\left(\frac{\alpha_1 V_1^{\,2}}{2g}\right)\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.28)$$

Where: h_o = contraction or expansion head loss (m)

C = contraction or expansion coefficient

2.5.1.2. HEC-RAS Parameters

HEC-RAS uses a number of input parameters for hydraulic analysis of the stream channel geometry and water flow. These parameters are used to establish a series of cross sections along the stream. Each cross section was divided into segments of left floodway, main channel, and right floodway, as illustrated in Fig 2.8.

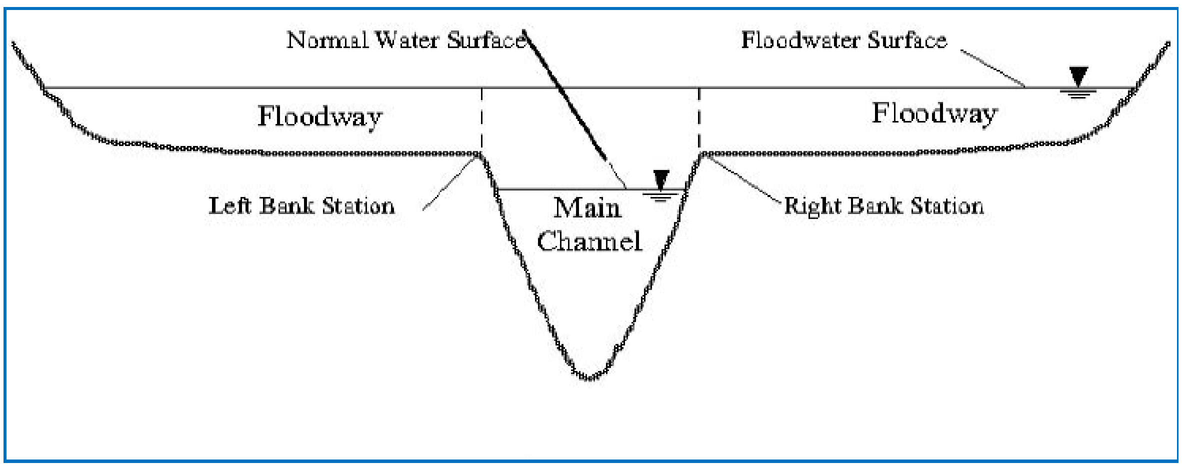


Figure 2-8 HEC RAS Stream Cross Section. Source: (US Army Corps of Engineers, 2010)

At each cross-section line, HEC-RAS assumes that energy is constant and that the velocity vector is perpendicular. As such, care should be taken to ensure that the flow through each selected cross section meets these criteria. After defining the stream geometry, flow values for each reach within the river system are entered. The channel geometric description and flow rate values are the primary model inputs for the hydraulic computations (Brunner, 2010).

2.5.2. Hydraulic Modeling Delineation of Flood Prone Area

Two models HEC-GeoRAS and HEC-RAS were used one after another for the Flood Inundation map. (i.e. first HEC-GeoRAS then HEC-RAS then back to HEC-GeoRAS) to accomplish the task (Solaimini, 2012). HEC-GeoRAS is a set of procedures, tools, and utilities for processing geographic information systems (GIS) data in Arc GIS version 10.3, using a graphical user interface (GUI). The interface allows preparation of geometric data for import into HEC-RAS and generation of GIS data from exported HEC-RAS simulation results. Automated GIS processing procedures in HEC-GeoRAS provides a valuable and expeditious method for repetitive hydraulic model development during floodplain analysis.

HEC-GeoRAS Version 10.3 was used to extract cross-sectional station-elevation data from a digital elevation model (DEM) represented by a triangulated irregular network (TIN). Downstream reach lengths and bank station locations were determined for each cross section. The automated procedures for extracting geometric data proved consistent and efficient for the development of floodplain models. The geometric data was imported into HEC-RAS Version 5.0.1 using a data exchange format developed by HEC-GeoRAS. The resultant water surface elevations exported from HEC-RAS simulations were processed by HEC-GeoRAS for floodplain delineation and water depth calculations. Analysis of cross-section exported from HEC-RAS was also performed using HEC-GeoRAS (Merwade, 2010). Pre-GeoRAS creates a series of line themes pertinent to developing geometric data for HEC-RAS. The RAS themes created are the Stream Centerline, Flow Path Centerlines, Main Channel Banks optional, and Cross Section Cut Lines referred to as the RAS Themes.

HEC-GeoRAS was an Arc GIS extension that provides with a set of procedures, tools, and utilities for the preparation of GIS data for import into HEC-RAS, and generation of GIS data from RAS output. Terrain TIN (a triangulated irregular network) using Arc GIS extension HEC-GeoRAS. The application of computer technology to analysis of the rainfall-runoff process and the hydraulics of natural rivers was greatly expanded. Hydraulic analysis was performed to predict flow characteristic and water surface elevations in the Omo-Gibe River. This hydraulics analysis uses the US Army Corps of Engineers computer program HEC-RAS, HEC-GeoRAS Extension and GIS application. Results showed that integration of HEC-RAS with GIS using HEC-GeoRAS Extension provides an effective environment for both hydraulic analysis and mapping. Traditional methods for Hydraulic analysis and delineating flood plain should be replaced with new technological models ((Roshun, 2011).

2.5.3. Pre-RAS (HEC-GeoRAS) Processing

The goal of this section was to develop the spatial data required to generate a HEC-RAS import file with a 3-D stream network and 3-D cross sections defined. The process was divided in three steps: Preparation of 3-D polyline themes defining stream centerline, cross-sections, stream banks, and flow path lines the second Use of the HEC-GeoRAS pre-RAS menu functions to extract 3-D spatial data from the TIN to develop 3-D polyline themes of the previously defined stream centerline, cross-sections, stream banks and finally flow path lines and Generation of the HEC-RAS Import File (Akerman, 2002).

2.5.4. Generating the RAS GIS Import File

To generate the RAS GIS Import File, the 3D stream Centerline and Cross Section Surface Line (3D) shape file created from the RAS Theme. Geometric data from the two 2D (stream Centerline and Cross Section Surface Line) shape files is written to the RAS GIS Export File.

The geometric data includes: river, reach, and station identifiers, cross-section cut lines, cross-section surface lines, main channel bank stations, downstream reach lengths for the left overbank, main channel and overbank. HEC-GeoRAS is an Arc GIS extension specifically designed to process geospatial data for use with the Hydrologic Engineering Center's River Analysis System (HEC_RAS). This extension allows to Create an HEC-RAS import file containing geometric attribute data from an existing digital terrain model (DEM) and complementary data sets. Process results exported from HEC-RAS. HEC-GeoRAS also enables viewing of exported results from RAS. The import file is created from data extracted from data sets (ArcGIS shape files) and from a Digital Terrain Model (DTM) represented by a triangulated irregular network (TIN). Prior to performing hydraulic computations in HEC-RAS, the geometric data must be imported and completed and flow data must be entered. Once the hydraulic computations are performed, exported water surface and velocity results from HEC-RAS may be imported back to the GIS using HEC-GeoRAS for spatial analysis. GIS data is transferred between HEC-RAS and Arc GIS main view using a specifically formatted GIS data exchange file (Asadi, 2013).

CHAPTER THREE

3. MATERIALS AND METHODS

3.1. Description of Study area

3.1.1. Location

The Omo-Gibe river basin is one of the major river basins in Ethiopia and is situated in the southern part of the country. It lies between 4˚00'N & 9˚22'N latitude and between 34˚44'E &38˚ 24'E longitude and covers an area of 79,000 km^2 with a length of 550 km and an average width of 143.636 km. It is an enclosed river basin that flows in to the Lake Turkana in Kenya which forms its southern boundary. The western watershed is the range of hills and mountains that separate the Omo-Gibe Basin from the Baro-Akobo Basin. The north and northwest, the basin is bounded by the Nile Basin with small area in the northeast bordering the Awash Basin. The whole of the eastern side borders the Rift Valley Lakes Basin (Richard Woodroofe , 1996).

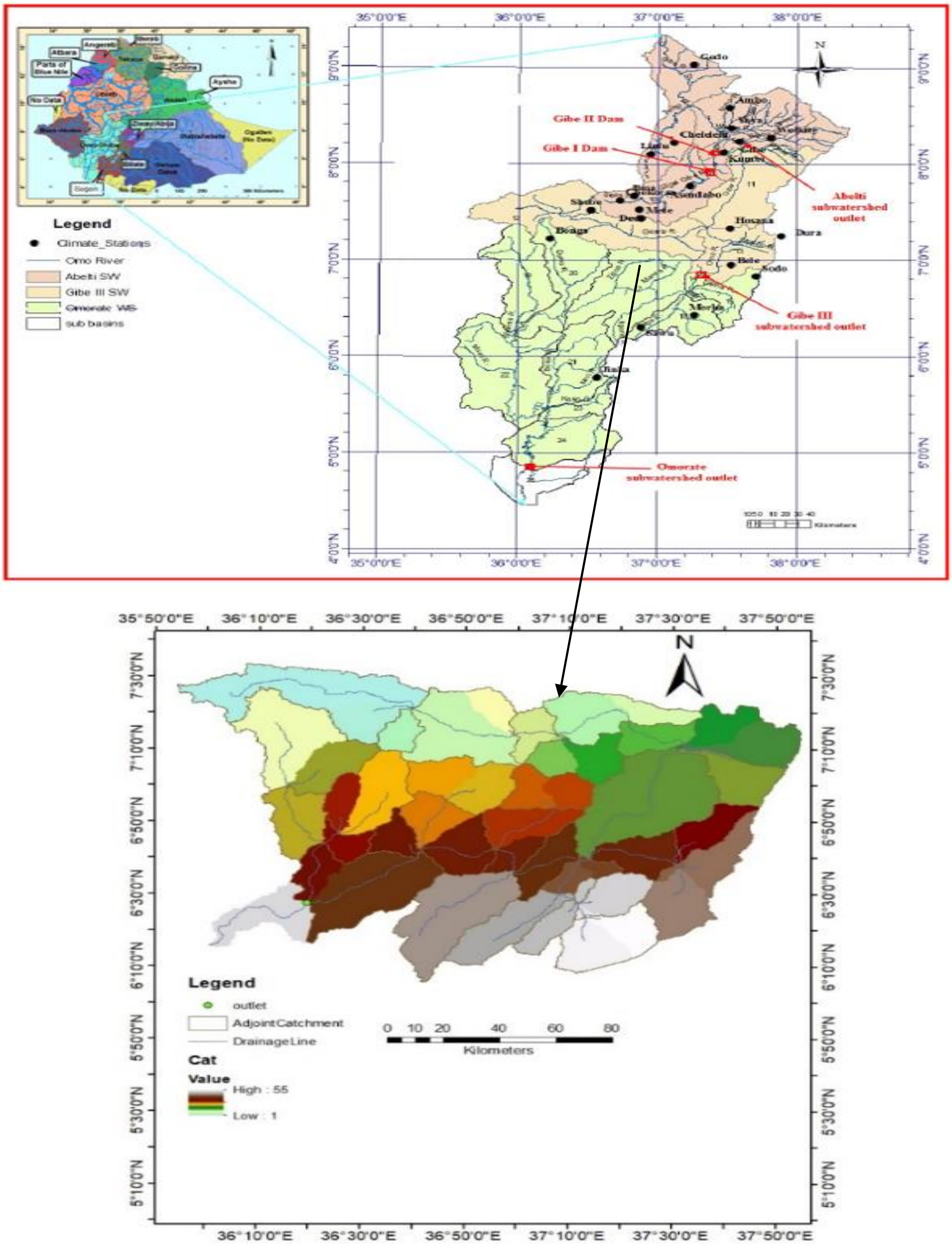


Figure 3-1 Location map of Omo Gibe River basin

3.1.2. Topography and slope

The topography of the Omo-Gibe basin as a whole is characterized by its physical variation, especially the difference between the northern two thirds which has mountainous to hilly terrain cut by the deeply incised gorges of the Omo, Gojeb and Gilgel Gibe rivers and the southern one third which is alluvial Plain punctuated by hilly areas. South from Hosaina towards Wolaita with rolling dissected upland plateau forming the highland which is being eroded into by the Omo and Gibe gorges exposing the underlying basalts. Between these highlands and the gorge, itself the landform is one of an undulating to rolling plain often exhibiting severe erosion, cut by deeply incised rivers. Most slopes are Greater than 10% and the altitude range is 1650-1850 m.

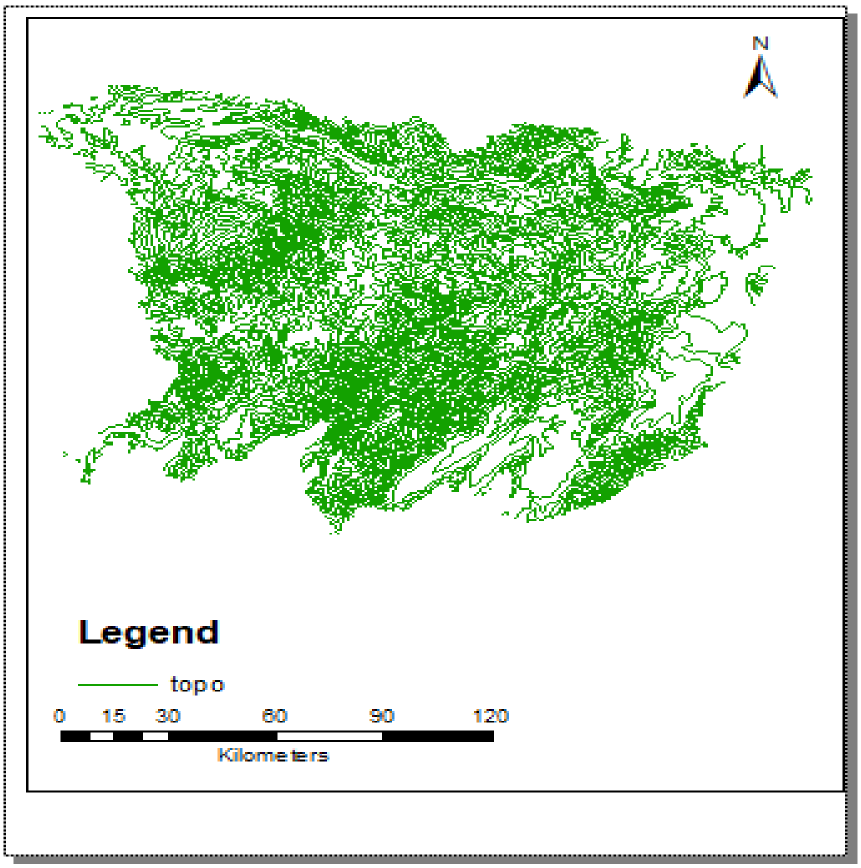


Figure 3-2 Topography of the Lower Omo Gibe Catchment

The central part of the Basin, north and west of the Omo gorge, south of the Gibe Gorge and around Jima, Bonga, Chida, Waka and south to Berehuma, is composed primarily of high relief hills and mountains of 2000-3300 m. North of the Gojeb River the upland areas are relatively undissected but to the south the Kulo Konta hills are severely dissected. These hills are interspersed with relatively narrow undulating upland basins with altitudes of 1800-2000m along the Gilgel Gibe east of Jima and on the upper Gojeb west of Jima. The lower Omo plain is characterized by the fact that it is a relatively recent feature formed by successive movements of Lake Turkana and, though it is relatively flat, it contains numerous beach ridges and depressions. The Omo delta, in the south of the Basin and at the northern end of Lake Turkana has aggraded to such an extent in recent years only some 25km^2 of the lake is in Ethiopia.

3.1.3. Land use and soil

Land use of the flood plain is mainly dominated by cultivated land. Most of the study area is characterized by cultivated lands as most of the area is rural. The catchment is dominated by Lithosols, Dystric Cambisols and eutric Nitosols. The three soil types cover almost 50% of the area. Orthic acrisols are another type of soils which dominated the catchment.

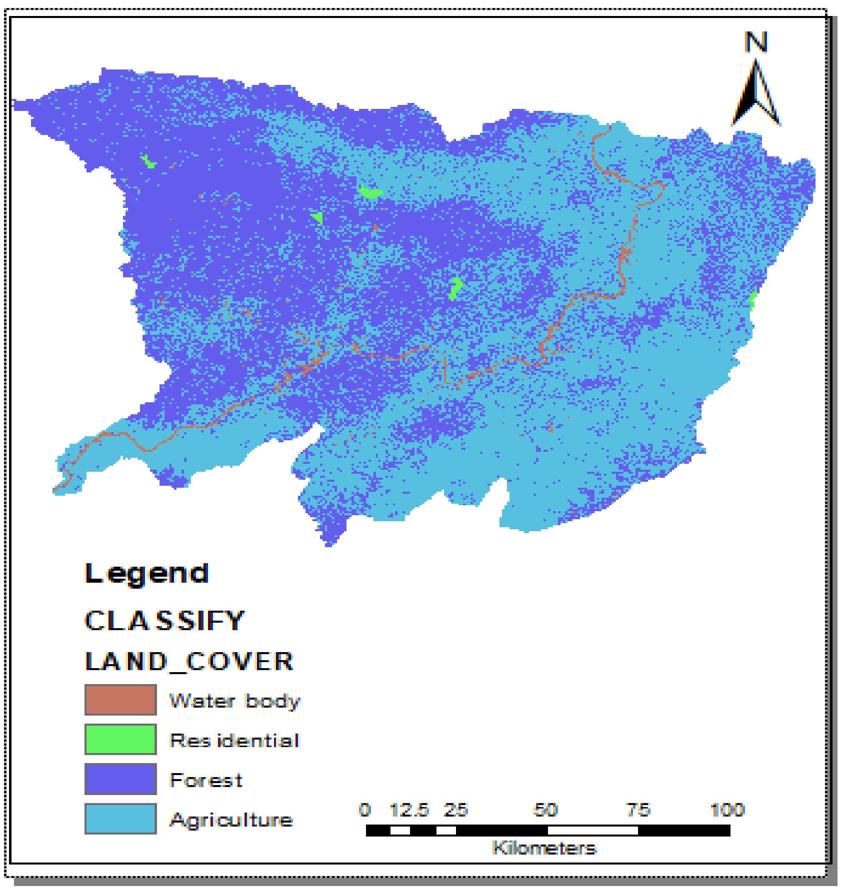


Figure 3-3 Land use map of the Lower Omo Gibe catchment

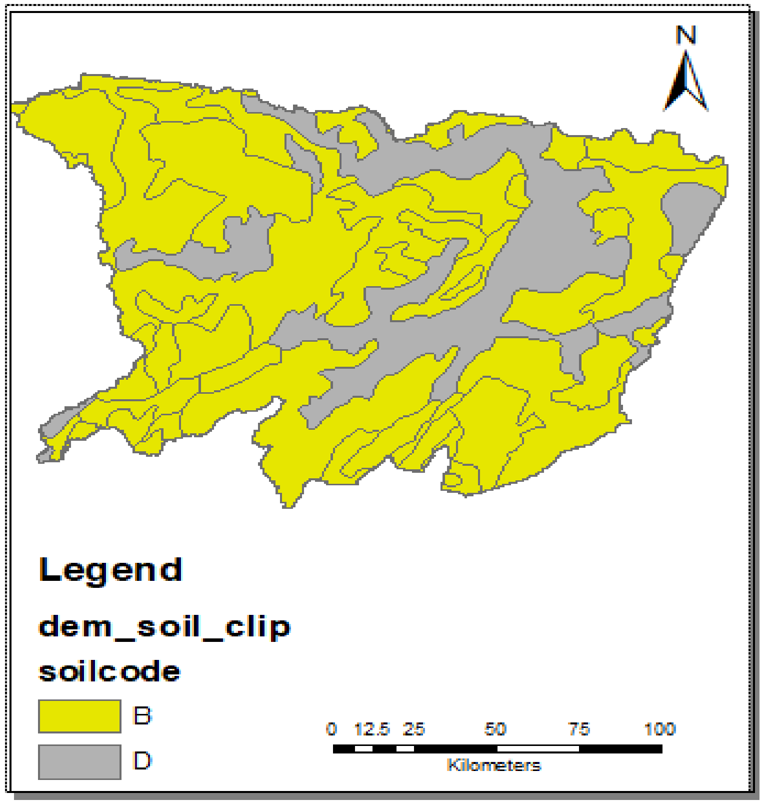


Figure 3-4 Soil type of the Lower Omo Gibe catchment

3.1.4. Rainfall distribution

In terms of rainfall the Basin can be split in to four regions, three of them having a unimodal and one is a bimodal rainfall regime. The northern part of the basin, including Bako, Weliso, Welkite and south to just north of Jima, has rainfall for about seven months, from March to September with a range of 1100-1800 mm per annum. The small rains are from March to May and the main from June to September with a marked increase in July and August.

The north-center area, including Bonga, Jima and Wolaita, has a more even distribution of rainfall over March to September with peak in July and august. The region generally receives more than 1200mm, rising to 2000 mm on the western fringes (outside the main stream) north of Bonga.

The southern-center area, including Maji, Jinka and Sawula, has a prolonged rainy season of nine months. The amount varies from 600 mm in the lower valleys to about 1800 mm in the hilly areas around Maji and in the west. Field observations and local experience indicate that around Sawula, at least, the rainfall pattern is different to the north with the major rains in February and March rather than later in the year. The evidence for this prolonged rainy season was confirmed by field experience with wet conditions experienced from March through November.

Generally according to the isohyetal map of the OGMP (Omo Gibe Master Plan), the rainfall ranges from over 1900 mm/year in the northern and western part of the basin, to about 300 mm/year in the lower part of the basin near Lake Turkana.

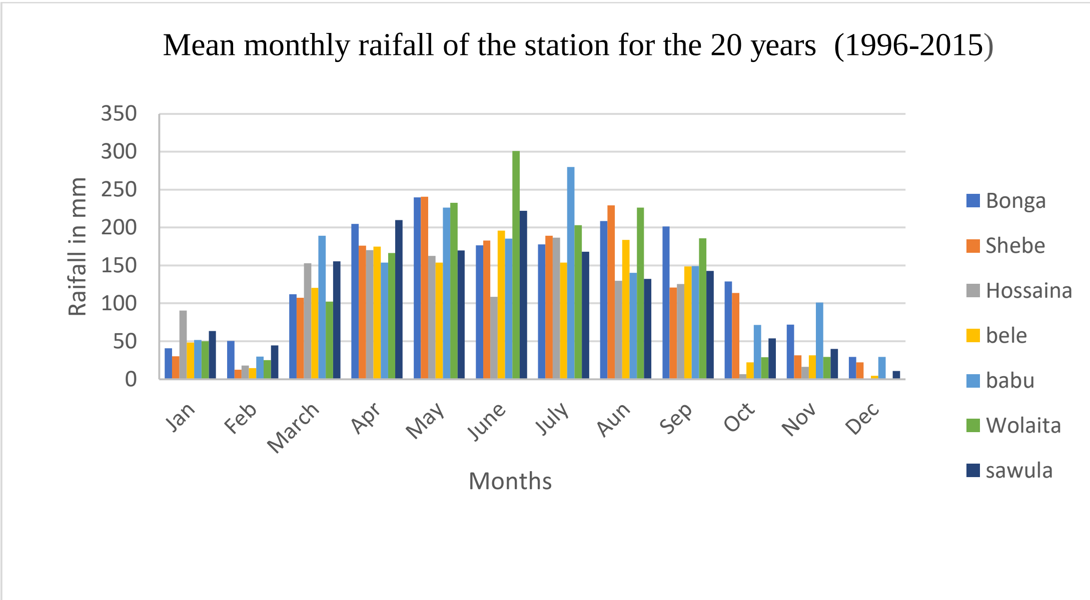


Figure 3-5 Mean monthly rainfall of the Lower Omo Gibe catchment

3.2. Materials and Tools

The materials indispensably used in conducting of this study were; literatures, internet, etc. And the tools used to undertake the research were software. The general descriptions of the tools are described here under in Table 3-1.

Table 3-1 Description of tools

R.no	Tools	Descriptions
1	ArcGIS, Arc hydro	For delineation of streams to be used as an input for HEC-GeoHMS and they are terrain preprocessing
2	Easy Fit software	To know best fit flood Probability distribution
3	HEC-GeoHMS and HEC-HMS	Data processing for Watershed function and for generation of Basin Model file and importing it in to HEC-HMS and to determine peak flow, Time of peak, Volume of discharge on the study area.
4	HEC- GeoRAS and HEC-RAS	To digitize the RAS themes and/or flood Inundation mapping(Hydro-dynamic)
5	Google Earth	To Create kml for HEC-GeoRAS and GIS
6	Global Mapper	For data prepare such as shape file

3.3. Flow Chart of the Thesis

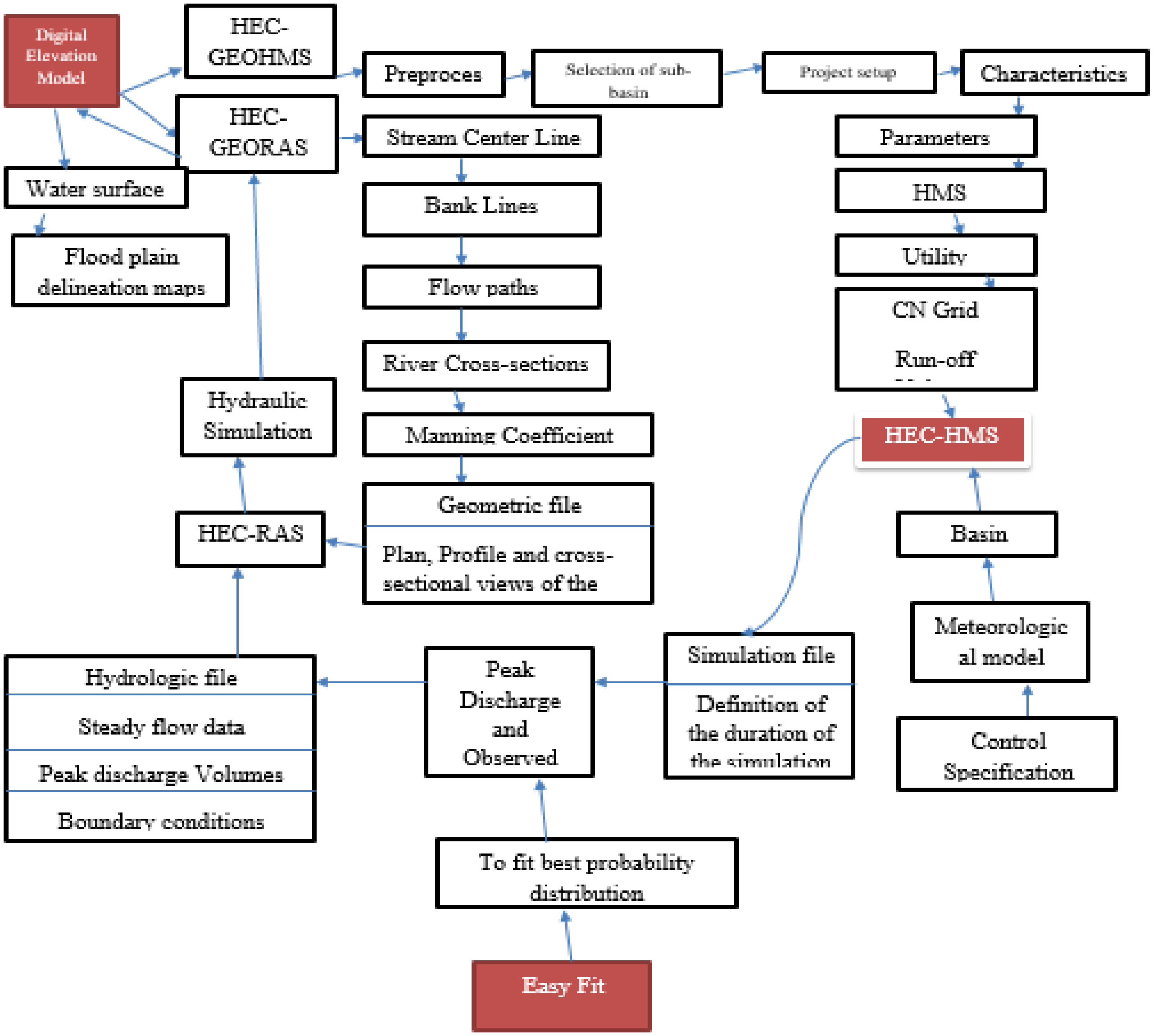


Figure 3-6 General Frame work of the Thesis

3.4. Data Collection and Analysis

3.4.1. General

Three basic data sets are necessary for the modeling work. These are the meteorological data (rainfall and temperature), the hydrological (stream flow) data and the digital elevation model (DEM 30m resolution: source ASTER).

The HEC-HMS and Easy fit model requires meteorological and hydrological input data in a daily time step including rainfall, temperature and daily flow data at the sub-basin outlet. Daily areal rainfall, mean, maximum and minimum temperatures data were compiled for all available stations in the basin. Most of the meteorological stations for which data were collected are located inside the basin and some are located around it. 20 years was collected from ministry of water irrigation and electricity. Considering the length of record, continuity of data, concurrent period of observation and the distribution of stations in the sub-basin, eight or Seven meteorological stations are selected for the study. The distribution of these stations within the sub-basin is not even. Hence, the Thiessen Polygon method was used to estimate the areal rainfall and minimize the error introduced by spatial variability.

3.4.2. Meteorological data

The First most important time series data necessary for this research were rainfall data. The source of raw meteorological data in Ethiopia is the National Meteorological Service Agency (NMSA). A request for daily rainfall data of 20 years up to 30 years of period in addition daily mean temperature, relative humidity and sunshine hour duration data was made to the agency. Following the approval of the agency's higher official, daily data of up to 20 years' period including many more stations that are not exactly used in the model work were collected. Out of the entire available automatic recording stations those which are in or proximate to the watersheds considered for the research work were selected. Accordingly, a total of six rainfall stations were selected for use in the research work. The location of these rainfall stations is shown in Figure 3-5.

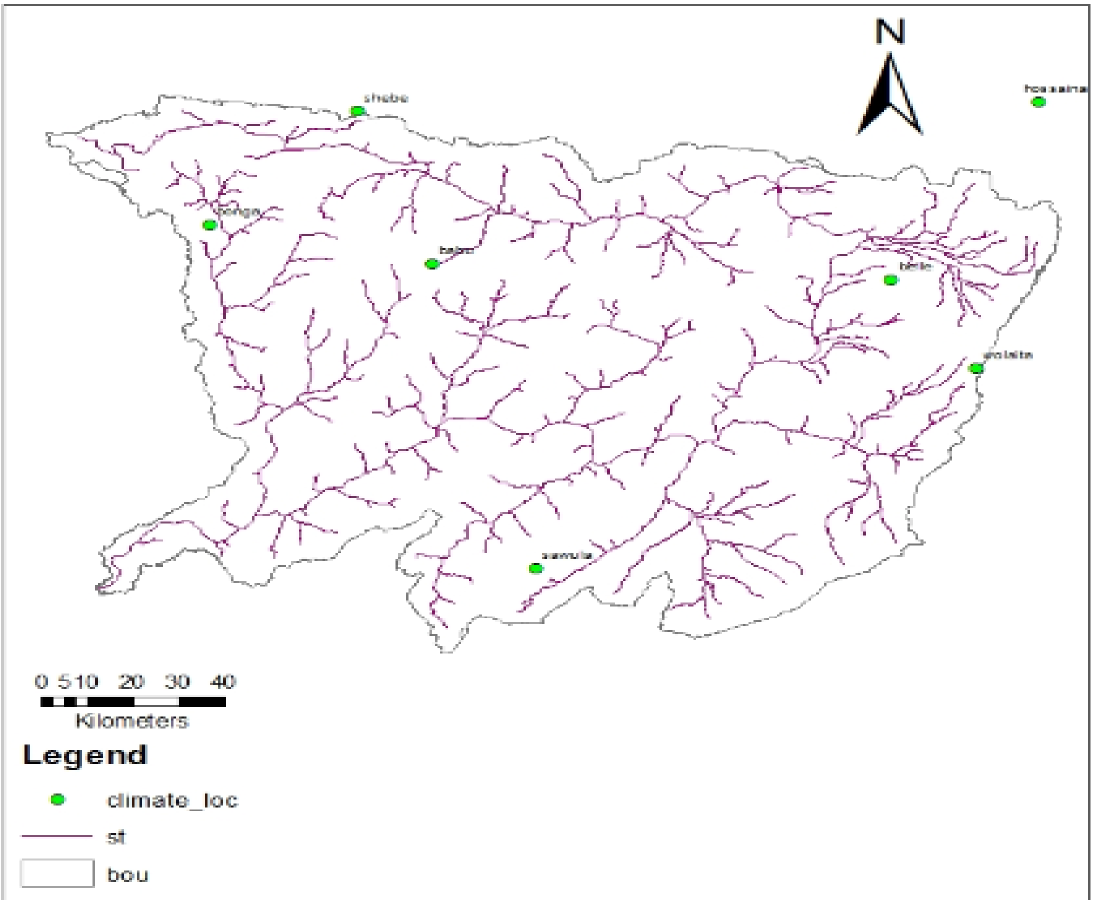


Figure 3-7 Locations of Rainfall Gauging stations by Arc-hydro

Table 3-2 Summary of the rainfall stations

s.no	Station name	Latitude(degeree)	Longitude(degree)	Elevation(m)	years of data used	Missed data	Remark
1	Bonga	7.50	36.50	1779	1996-2015	13.10%	Filled
2	Babu	7.88	36.78	1707	1996-2015	25.11%	Filled
3	Shebe	7.50	36.52	1813	1996-2015	33.13%	Filled
4	Wolaita	6.81	37.73	1854	1996-2015	4.63%	Filled
5	Dannal 2	6.59	37.55	1303	1996-2015	3.08%	Filled
6	Bele	7.74	40.00	2424	1996-2015	77.10%	Filled
7	Morka	6.42	37.31	1221	1996-2015	3.90%	Filled
8	Hossaina	7.57	37.85	2307	1996-2015	4.20%	Filled
9	Jinka	5.77	36.55	1373	1996-2015	2.30%	Filled

3.4.3. Evapotranspiration

Evapotranspiration was a collective term that includes evaporation from the land surface and evaporation from vegetation cover. Generally, it does mean that water removed from the watershed. Normally, HEC-HMS- includes three options to estimate evapotranspiration. These are the Penman-Monteith method (Penman, 1948),the Priestley-Taylor method (Priestley, 1972) and the Hargreaves method (Hargreaves, 1975). One of the three methods were selected to calculate the potential evapotranspiration from the watershed depending up on the data available. The model was also read if a separate daily PET (Potential Evapotranspiration) values are applied for potential evapotranspiration method. The data requirements for the application of these three PET methods are very different. The Penman- Monteith method requires solar radiation, air temperature, relative humidity and wind speed. The Priestley-Taylor method requires solar radiation, air temperature and relative humidity. The Hargreaves method requires air temperature only. The Hargreaves-Samani equation is expressed as (Allen et al., 1998):

$$E_{to}=0.023\ R_a\ (T_{mean}+17.8)\ *(T_{max}-T_{min})^{0.5} \qquad (3.1)$$

Where: E_{to}=Evapotraspiration and R_a=Extraterrestrial Radiation.

Tmax is the daily maximum temperature (ºC); Tmin is the daily minimum temperature (ºC).

$$R_a=\frac{24(60)}{\pi}\ G_{sc} d_r\ [\omega_s\ \sin(\varphi)\sin(\delta)+\cos(\varphi)\cos(\delta)\sin(\omega_s)] \qquad (3.2)$$

$$\omega_s=\arccos[-\tan(\varphi)\tan(\delta)] \qquad (3.3)$$

$$d_r=1+0.033\cos\left(\frac{2\pi}{365}\ J\right) \qquad (3.4)$$

$$\delta=0.409\sin\left(\frac{2\pi}{365}\ J-1.39\right) \qquad (3.5)$$

3.4.4. Estimating Missing Precipitation

Before using the rainfall records of a station, it was necessary to first check the data for continuity and consistency. The continuity of a record may be broken with missing data due to many reasons such as damage or fault in a rain gauge during a period. The missing data can be estimated by using the data of the neighboring stations. A number of methods have been proposed for estimate missing rainfall data (WMO, 2008). The station average method is the simplest method. The normal- ratio and quadrant methods provide a weighted mean, with the former basing the weights on the mean annual rainfall at each gauge and the latter having weights that depend on the distance between the gauges where recorded data are available and the point where a value is required.

The station average method for filling missing data is conceptually the same as the simple weight method for estimating a mean daily precipitation. This method may not be accurate when the total annual rainfall at any of the n region gauges differs from the annual rainfall at the point of interest by more than 10%.

The normal-ratio method was conceptually simple; it differs from the station-average method of that the average annual rainfall was used in deriving weights. If the total annual rainfall at any of the m region gauges differs from the annual rainfall at the point of interest by more than 10%, the normal- ratio method is preferable. Because this method is more advanced than station average method and simple, I considered this method for filling missed rainfall data in this study.

The general formula for computing P is

$$\overline{\mathrm{P}} = \frac{\mathrm{N_X}}{\mathrm{M}}\left(\frac{\mathrm{P_1}}{\mathrm{N_1}} + \frac{\mathrm{P_2}}{\mathrm{N_2}} + \frac{\mathrm{P_3}}{\mathrm{N_3}} + \frac{\mathrm{P_4}}{\mathrm{N_4}} + \ldots\ldots + \frac{\mathrm{P_m}}{\mathrm{N_m}}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6)$$

Where:

$\mathrm{N_X}$= Average annual precipitation at the missing data.

$\mathrm{N_{1,}}\ \mathrm{N_2},\mathrm{N_{3,}}\ \ldots\ldots\mathrm{N_m}$ =Average annual precipitation at the adjacent site.

The selected, available and missed rainfall data statistics are shown in the Table and also simple weight method requires rainfall working gages to be within 10% were used to fill the missed data by using the following equation.

$$\widehat{\mathrm{P}}=\frac{1}{n}\sum_{\mathrm{i}=1}^{\mathrm{n}}\mathrm{p_i}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.7)$$

3.4.5. Homogeneity of the stations

In order to select representative meteorological stations for the analysis of areal precipitation, checking homogeneity of group stations is essential. The homogeneity of the selected gauging stations monthly rainfall records has been carried out by non-dimensional zing using equation:

$$\mathrm{P_i}= \frac{\overline{\mathrm{P_\imath}}}{\overline{\mathrm{p}}}*100\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.8)$$

Where:

$\mathrm{P_{i=}}$Non-dimensional Value of PP for the month i

$\overline{\mathrm{P_\imath}}$=Over years averaged monthly precipitation for the station i

$\bar{P}$=The over years average yearly precipitation of the station and plotted to compare the stations.

3.4.6. Determination of Areal Rainfall

The According to Thiessen, the average rainfall, R_{areal} over the area can be computed from:

$$R_{areal}=\sum_{i=1}^{n}\frac{Ri\ Ai}{At} \quad \ldots\ldots 3.9$$

Where, R_i is the rainfall at station i

A_i is the polygon area of station i

A_t is total catchment area, and n is the number of stations.

The area functions $\frac{A_i}{A_t}$ are known as the Thiessen coefficients(weight) and once they are determined for a given stable station network, the areal rainfall can be computed for the set of rainfall measurements (AlHallaq, 2008).

3.4.7. Hydrological Streamflow Data

Stream flows gauging stations in the Omo Gibe River Basin are mainly maintained by the hydrology department of the Ethiopian Ministry of Water, Irrigation and Electricity (EMWIE) which process and archive data. But many of these gauges are situated in the upper part of the basin and none in the Middle and lower Omo. Most of the time, long term data for the design and planning of water resources projects have not always been obtainable and also records of hydrological data are usually short and often have breaks in the records. Hence, the hydrological flow data of the river was produced by the following methods. The method could be by filling the gaps of the directly measured streamflow gauging stations using multiple regression of R-program by correlating with the nearest stations. But, most of the time Rivers are not gauged at the confluence point. The site of water resource development in any of the uses can be at or ungauged site. If gauged data with sufficient record length is available, then such data were used. In the absence of sufficient record length, or for completely un gauged areas, data extension and generation can be employed (Awlachew, 2000).

To determine the overall discharge at the confluence of Omo-Gibe River, stream flow data was transferred to the site of interest using area ratio methods and convolution equation. The recommended guide lines for area ratio method to assess the available dependable flow for the potential assessment purpose is: -

$$Q_{Ungauge}=\left(\frac{A_{Ungauge}}{A_{gauge}}\right)^{n} *Q_{gauge} \quad \ldots\ldots (3.10)$$

Where:

$Q_{Ungauged}$= discharge at the site of interest

Q_{gauged}= Discharge at the gauge site

$A_{Ungauged}$=Drainage area at the site of Interest

A_{gauged}=Drainage area at the gauging site

n- Varies between 0.6-1.2

If the $A_{Ungauge}$= is within 20% of the A_{gauge}($0.8 \leq \frac{A_{Ungauge}}{A_{gauge}} \leq 1.2$) then n=1 to be used. The estimated discharge at the site was within 10% of actual discharge (Awlachew, 2000).

When $A_{Ungauge}$= is within 50% of the A_{gauge}, two stations data are considered for data transferring. Relation can be developed to estimate a weighted average flow at a site lying between upstream and downstream gauges.

$$Q_{Ungauge}=\frac{(A_{gauge1}-A_{Ungauge})*Q_{gauge}+(A_{Ungauge}-A_{gauge2})*Q_{gauge2}}{A_{gauge1}-A_{gauge2}} \ldots\ldots. (3.11)$$

Where: -

Gauge1 upstream gauging site and

Gauge2 downstream gauging site.

All flows below Abelti Gauging station were transferred from gauged station to Gibe and Omo-Gibe rivers confluence according to the above given criterion (equation 3.11) and formula. So, these flow data were the main inputs for the simulation model.

Table 3-3 Regression equations used for filling of missed hydrological data

Missed River stations (Y)		Nearby River Stations (X)		Correlation coefficient (r^2)	Equation	Remark
River Name	Id	River Name	Id			
G.gibe@Abelti	061015	G.gibe Nr.Assendabo	091008	0.85	Y=2.47X+77.28	Filled
G.gibe Nr.Assendabo	091008	Gojeb @shebe	091012	0.89	Y=0.78X+0.53	Filled
Gojeb@ shebe	091012	G.gibe Nr.Assendabo	091012	0.89	Y=1.02X+14.69	Filled
Demie River at Oreta alem	092011	MazieNr. Morka	092008	0.633	Y=9.81X+0.61	Filled

3.5. Best Fit flood probability distribution

In order to describe the amount of maximum yearly observed data, it was necessary to identify the distributions, which best fit to the data. In this study, around seven continuous probability distribution with Normal, Lognormal, Log-Pearson type III, Gamma, Weibull, Inverse Gaussian and Generalized Extreme value distribution are considered to test the goodness of fit, but only four distributions were used for the discharge comparison with the HEC-HMS (Hydrologic Engineering Center-Hydrologic Modeling System). The analysis of observed data was prepared with the help of Easy Fit software and Microsoft Excel.

3.5.1. Easy Fit Software

Easy Fit is a data analysis and simulation software which enables us to fit and simulate statistical distributions with sample data, choose the best model, and use the obtained result of analysis to take better decisions. This software can function as a stand-alone windows application or as an add-on for Excel spread sheet.

3.5.1.1. Excel integration

Easy Fit program is easily integrated in main menu of Excel and allows to implement the analysis and simulation in Excel environment. Easy fit software benefits of more than 650 spread sheets in Excel which can facilitate the calculation tasks. With Easy Fit program to analyze large datasets (up to 250,000 data points), Calculate the graphical statistics and Organize the data and enter obtained results in your project file. See the graph and parameter in Appendix- H.

3.6. Terrain Processing using Arc Hydro and HEC-GeoHMS

3.6.1. HEC-GeoHMS

HEC-GeoHMS is a set of ArcGIS tools specifically designed to process geospatial data and create input for the HEC-HMS. HEC-GeoHMS provides the connection for translating GIS spatial information in to model files for HEC-HMS. The GIS capability is for data formatting, processing and coordinate transformation. Currently, HEC-GeoHMS operates on DEM to derive sub-basin delineation and to prepare a number of hydrologic units. HEC-HMS supports these hydrologic inputs as starting point for hydrologic modeling. In this paper it is intended to derive parameters like: Curve Number, Basin Lag, and Time of concentration and Loss.

3.6.2. Terrain Processing Using Arc Hydro

The first step in doing any kind of hydrologic modeling involves delineating streams and watersheds, and getting some basic watershed properties such as area, slope, flow length, and stream network density. With the availability of digital elevation models (DEM) and GIS tools, watershed properties can be extracted by using automated procedures. The processing of DEM to delineate watersheds is referred to as terrain pre-processing. There are several tools available online for terrain pre-processing. In this study, we were use Arc Hydro (tools version that works with Arc-GIS 10.3) to process a DEM to delineate watershed, sub-watersheds, stream network and some other watershed characteristics that collectively describe the drainage patterns of a basin. The results from terrain processing can be used to create input files for many hydrologic models using HEC- Geo HMS. All the steps in the Arc Hydro Terrain Pre-processing menu should be performed in sequential order, from top to bottom. The procedure followed for terrain processing using Arc Hydro were explained under using 30x30 DEM extracted for the respective sub basins and river feature class of the study area. For simplicity the main steps undertaken by arc hydro processing are: -DEM reconditioning, Fill sinks, Flow direction, Flow accumulation, Stream definition, Stream segmentation, Catchment grid delineation, Catchment polygon processing, Drainage line processing, Drainage point processing, longest flow path for the catchment and Slope determination.

3.7. HEC-HMS Model

Data models are central to the application of information technology because they are the means by which the real world is represented inside a computer. The model starts with data processing and acquisition of the HEC-HMS. The data required for the model is derived from different hydrological components. This hydrologic representation imported into HEC-HMS is then combined with precipitation data and control specifications to create flow and time series data for use in a Hydrologic Data Model HEC-HMS. The flow and time series data from HEC-HMS was imported into the hydraulic model HEC-RAS along with its geometry data to develop water surface profiles. To close the loop, data is then once again used in ArcGIS with a HEC-GeoRAS extension from HEC-RAS to create a visual model used to delineate floodplain.

3.7.1. Model setup

The main input data used for HEC-HMS are: areal precipitation, Evapo-transpiration, observed flow, base flow and different watershed characteristics obtained from different sources. An HEC-HMS simulation was defined by three components: The Basin Model, the Meteorological Model, and the Control Specifications. The Basin Model contains a schematic consisting of any combination of the six objects (sub-basin, reach, junction, source, sink and reservoir). The Basin Model stores information about the properties and connectivity of the objects in the schematic. In this research paper only the first three components are used. The Meteorological Model contains time series information consisting of rainfall and evaporation data. These data are associated with rain gages that the user defines in the Meteorological Model. The Control Specifications component defines simulation properties such as duration and time step. The HEC-HMS model for the lower Omo-Gibe catchment was done considering and dividing the sub-basin in to three sub-catchments. The calibrated model was used for runoff generation for different frequency storms. In this research paper the Specified Hyetograph method were selected. They are specified by using Theisen polygon created in ArcGIS 10.3.

3.7.1.1. Basin model

A general basin model consisting of sub-basin1, sub-basin2, and sub-basin3 were set up in HEC-HMS generated with ArcGIS for the study area. In addition to three sub-basins, an out let element was used in the basin model to relate the simulated flow to the historical observed total flow of the sub-basins. In this particular study for the respective sub basins, depending on the availability of time and data requirement, simulation was done with (SCS Curve number) method, SCS Unit Hydrograph, Muskingum model (K and X Coefficient) and Monthly constant base flow condition.

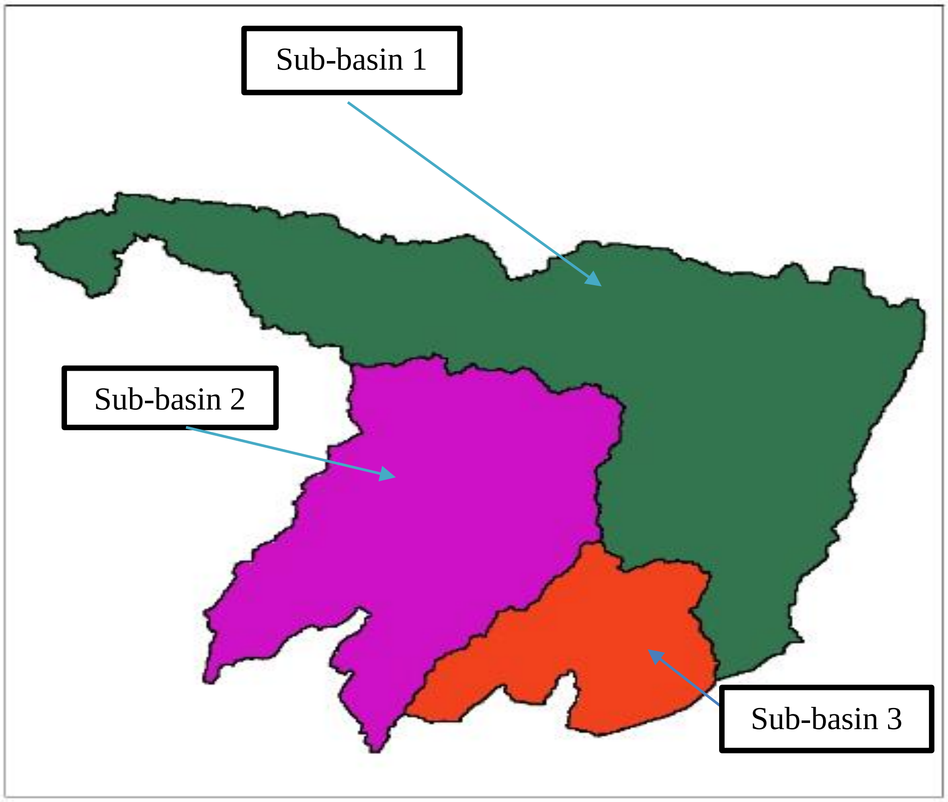


Figure 3-8 Sub-basins of the catchment of the study area

Table 3-4 Sub-basin area and contributing rainfall station

Sub-basin	Area(km2)	Contributing RF Stations
Sub-basin 1	10370	Shebe, Hossaina, Bele, wolaita
Sub-basin 2	6109.2	Babu
Sub-basin 3	2416.5	Sawula

There was a total of six stations used to represent the catchment. The stations are selected depending on their relative position to each sub-basin, data availability and the area they cover with respect to others.

3.7.2. HEC-HMS model components and processing steps

The hydrologic model was generated with the help of the HEC-GeoHMS (USACE, 2003) using DEM of the region. Using terrain data in the form of a DEM, HEC-GeoHMS, an extension of Arc GIS view creates HMS input files in the form of stream network, sub-basin boundaries, connectivity of various hydrologic elements etc. through a series of steps collectively known as terrain pre-processing and basin processing. The physical representation of watersheds or basins and rivers was configured in the basin model. Hydrologic elements were connected in a dendritic network to simulate runoff processes. The basin was divided into 3 sub-basins as shown in figure 3.12. Representing the main tributaries of the Omo-Gibe river. Then it was supplemented by following models.

3.7.2.1. Loss determination

The term loss refers to the amount of rainfall infiltrated into the soil. HEC-HMS supports the most common methods for calculating losses–such as initial/constant, SCS Curve No., gridded SCS Curve No., and the Green and Amps, and provides a moisture depletion option for simulations over extended periods of time. In this paper for Specified Hyetograph method, SCS Curve number loss method was chosen because it has been used for long term simulations and also it is “mature” model that has been used successfully in many studies throughout the US (USACE, 2003), easy to set up and use, and not too much demanding in terms of data where much is missing in the study area.

3.7.2.2. Transform method

Runoff transformations convert excess precipitation on a sub-basin to direct runoff at the sub-basin outlet. Again, HEC-HMS allows runoff transformation determinations using lumped or linear distributed approaches. Similarly, the SCS Unit Hydrograph model for direct runoff computation was chosen because of not only due to simplicity and minimum data requirements but also it gives a good performance than the other modules. In addition, it is suitable for conceptual models to transform rainfall to runoff process. The HEC-HMS is a conceptual model in which the process during simulation cannot be observed. It only gives the final output from the given input. The surface runoff calculations were performed using SCS Unit hydrograph method which requires Lag time to be computed for implementation. Sub-basin element conceptually represents infiltration, surface runoff, and subsurface processes interacting together. The actual infiltration calculations are performed by a loss method contained within the sub-basin. All of the possible loss methods in HEC-HMS conserve mass. That is, the sum of infiltration and precipitation left on the surface was always be equal to total incoming precipitation. Thus, effective rainfall was generated from the catchment loss and total rainfall on it.

3.7.2.3. Models of base flow

Base flow can be an important parameter in flood studies because it defines a minimum river depth over which additional runoff accumulates. Models that neglect base flow may under estimate water levels and therefore fail to identify inundated reaches (Knebl et al., 2005). In this study constant monthly values were used for base flow computations.

Table 3-5 Base flow of the sub-basin

Sub-basin	J	F	M	A	M	J	J	A	S	O	N	D
1	20.71	18.58	14.76	15.47	23.25	41.09	58.45	83.08	81.34	47.77	26.06	20.69
2	104.1	93.3	68.8	60.6	69.2	122.8	249	436.5	391.3	200.2	132.6	104.6
3	1.32	1.52	1.14	2.32	1.62	1.66	2.24	3.09	2.38	2.11	1.64	1.3

3.7.2.4. Meteorological model (precipitation)

The normal ratio method and simplest weight method (station average method) was method addresses dynamic data problems.

3.7.2.5. Routing method

Muskingum routing is used to route the channel for continuous hydrological modeling. Automated calibration (optimization) was found to give optimum and reliable model parameters. The objective function used for automated calibration (optimization) is the Peak-Weighted RMS Error. The HEC-HMS model has two optimization algorithms: Univariate gradient and Nealder Mead search algorithm. Univariate gradient was used for this study.

3.7.2.6. Control Specifications

The time span of a simulation was controlled by control specifications. Control specifications include a starting date and time, ending date and time, and computation time step. A computation run was created by combining a basin model, meteorological model, and control specifications.

The available records of 6 precipitation stations and one outlet stream flow gauge station were used for calibration and verification of the HEC-HMS model. The calibration was done using daily data for the period from 01 Jan 1998 to 31 Dec, 2007 (10 years) and also Validation was done using daily data for the period from 01 Jan 2008 to 31 Dec,2015(8 Years).

3.8. Model calibration and validation

Model calibration is the process of adjusting selected model parameters values and other variables in the model in order to match the model outputs with the observed values. The calibration procedure involves a combination of both manual and automated calibrations. The manual calibration proceeds the automate optimization to ensure a physically meaningful set of initial parameters generated from HEC-GeoHMS and Arc Hydro for the catchment. A total of 20 years' historical data from 1996 to 2015 was used, Warm up 2 Years data (1996-1997) and 10 years' data (1998-2007) are used for calibration. Model Validation is the process of testing the model ability to simulate observed data, Other than those used for the calibration, within acceptable accuracy. During this process, calibrated model parameter values are kept constant. The quantitative measure of the match is again the degree of variation between computed and observed hydrographs. The models are validated for a period of Eight years (2008-2015)

3.9. Sensitivity Analysis

Sensitivity analysis is a method to determine which parameters of the model have the greatest impact on the model results. There are three parameters (curve number, initial abstraction and lag-time) of the event model that were subject to the sensitivity analysis. The SCS curve number method, which is used to handle the infiltration loss in the sub-basins has three parameters such as curve number, initial abstraction and percent impervious area in the basin. Percent impervious area is taken as 0% since no urban settlements inside the sub-basin. Therefore, the remaining two parameters (curve number, initial abstraction) of SCS curve number method were Calibrated. The SCS unit hydrograph method which is used to model the transformation of precipitation excess into direct surface runoff, has lag time parameter. This parameter was calibrated as well.

3.9.1. Model efficiency/performance

The performance of a model must be evaluated on the extent of its accuracy, consistency and adaptability (Abushandi, 2013). A forecast efficiency criterion was therefore necessary to judge the performance of the model. Assessing performance of a hydrologic model requires subjective and/or objective estimates of the closeness of the simulated behavior of the model to observations. For the Omo Gibe catchment study, model simulation has been evaluated using efficiency criteria such as coefficient of determination (R^2) and [Nash Sutcliff (ENS), 1970]. The R^2 coefficient and ENS simulation efficiency measure how well trends in the measured data are reproduced by the simulated results over a specified time period and for a specified time step. The range of values for R^2 is 1.0 (best) to 0.0. The statistical index of modeling efficiency (ENS) values range from 1.0 (best) to negative infinity.

The Nash Sutcliffe (ENS) efficiency equation is given by

$$E_{NS}=1-\frac{\sum_{i=1}^{n}(q_{oi}-q_{si})^2}{\sum_{i=1}^{n}(q_{oi}-\overline{q_o})^2} \quad \text{(3.12)}$$

And the standard R^2 is given by

$$R^2=\frac{[\sum_{i=1}^{n}(q_{si}-\overline{qs})(q_{oi}-\overline{qo})]2}{\sum_{i=1}^{n}(q_{si}-\overline{qs})^2\ \sum_{i=1}^{n}(q_{oi}-\overline{qo})^2} \quad \text{(3.13)}$$

Where: qo, i = observed value at the i th time interval

qs, i = simulated value at the i time interval

qo, av = average value of the observed discharge

qs, av =average value of the simulated discharge

n =Number of sample data

3.10. Modeling by frequency storm method

With the input from HEC-GeoHMS and some edition from the main HEC-HMS, the model was simulated for rainfall intensity of 2, 10, 50, and 100-year return periods. The frequency intensity values are found from the Ethiopian Roads Authority drainage manual (ERA, 2013) from the Appendix G.

Table 3-6 IDF table for the study area (ERA Drainage Design Manual, 2013).

D(hr)	RF depth with return period			
	2	10	50	100
1	37.90	51.05	62.35	67.20
2	44.47	59.89	73.15	78.84
3	47.59	64.1	78.29	84.38
6	52.0	70.04	85.54	92.19
12	55.63	74.93	91.52	98.63
24(One day)	58.87	79.29	96.84	104.37

3.11. Hydraulic Flood Modeling

3.11.1. General

Flood models are the representation of the hydrologic and hydraulic processes in the water shed, river channel and floodplain. Accurate representation of the actual process is of paramount significance in predicting flood extent and depth. Determining the variation of flow characteristics in spatial and temporal resolution enables to design flood evacuation plan quite efficiently.

3.11.2. Development of DTM

The key data element that GeoRAS uses to develop the input data were terrain data, commonly referred to as a Triangular Irregular Network (TIN). One source of data used to develop TIN was a Digital Elevation Model (DEM). The TIN of the study area was developed from the 15m interval contour which is developed from DEM of the study data using the 3D-special analysis extension in the Arc GIS of the study area and is clipped using the Arc Tool Special Analysis.

DEM exist in grid (raster cell) format which can be displayed within ArcGIS, if the proper extensions are installed. The quality of this data is based on its resolution, or cell size. The smaller the cell is, the greater the resolution and accuracy. However, the smaller the cell size, the greater the memory and computation requirements. The usefulness of DEM for developing terrain models should be determined based on the cell size and the level of hydraulic analysis to be performed. The more approximate the analysis is to be, the greater the cell size that may be used. This can best be represented by TIN of the study area.

3.12. HEC-GeoRAS

HEC-GeoRAS is an Arc GIS extension specifically designed to process geo-spatial data for use with the Hydrologic Engineering Center River's Analysis System (HEC-RAS). The extension allows users to create RAS layers an HEC-RAS import file containing geometric attribute data from an existing digital terrain model (DTM) and complementary data sets. Water surface profile results may also be processed to visualize inundation depths and boundaries. HEC-Geo RAS extension for Arc GIS used an interface method to provide a direct link to transfer information between the Arc GIS and the HEC-RAS. The model uses the geometric attribute data from an existing digital terrain model (DEM) in TIN format and exported results from HEC-RAS model.

3.13. HEC-RAS Hydraulic Model

HEC-RAS is a hydraulic model developed by the Hydrologic Engineering Center (HEC) of the U.S. Army Corps of Engineers. The model is used for determination of water surface profiles for different flow scenarios. The peak discharge generated by the HEC-HMS model is used to determine the flow profiles and flood plain profiles for the selected flood return periods. HEC-RAS is intended for steady flow water surface profile computations and unsteady flow simulation. The system is capable of modeling subcritical, supercritical, and mixed-flow regimes for streams consisting of a full network of channels, a dendrite system, or a single river reach. The key data used in this model are DEM and the peak flow resulted from simulation of the HEC_HMS model.

The HEC-RAS program, like the other software's, it can be downloaded free of charge from the Hydrologic Engineering Center's. Hydrologic Engineering Center's River Analysis System (HEC-RAS) is the software predominately used in the field of hydraulic analysis for floodplain delineation. HEC-RAS, combined with Hydrologic Engineering Center's Geographical River Analysis System (HEC-GeoRAS), offers engineers a powerful tool in the process of hydraulic modeling and analysis. For each HEC-RAS project, there are three required components: The Geometry data, Flow data (Steady and unsteady), and Plan data. The Geometry data, for instance, consists of a description of the size, shape, and connectivity of stream cross-sections. Likewise, the Flow data contains discharge rates. Finally, Plan data contains information pertinent to the run specifications of the model, including a description of the flow regime. Each of these components is explored below individually.

3.13.1. Input data and model components

One of the functions of the HEC-RAS program is to determine surface elevations at any point of interest. The data needed to perform these computations are separated into geometric data and steady flow data (boundary conditions). The input data for HEC-RAS is imported from ArcGIS which was discussed in chapter four

3.13.2. Entering Flow Data and Boundary Condition

The discharge values for different return periods can be entered manually for steady flow. The roughness coefficients (Manning's coefficient) and boundary conditions were added to the model manually. The values selected were 0.055 and 0.041, for the stream channel and overflow banks, respectively. The model was run for Mixed flow regime conditions and steady flow water surface profile computations. The iterative solution of the energy equation, using the standard step method, solved the steady flow, while Manning's equation and contraction/expansion coefficients determined head losses.

Before applying the computation process the model must be set up for boundary condition. There are various methods of boundary condition used. The method used in this paper is the Normal depth at the downstream end of the reach. The model calculates the depth from the given elevation data and discharge. The water surface can also be used if the accurate and up-to-date value is available. Finally, the plan must be established for each model simulation. The plan has a user specified description and application.

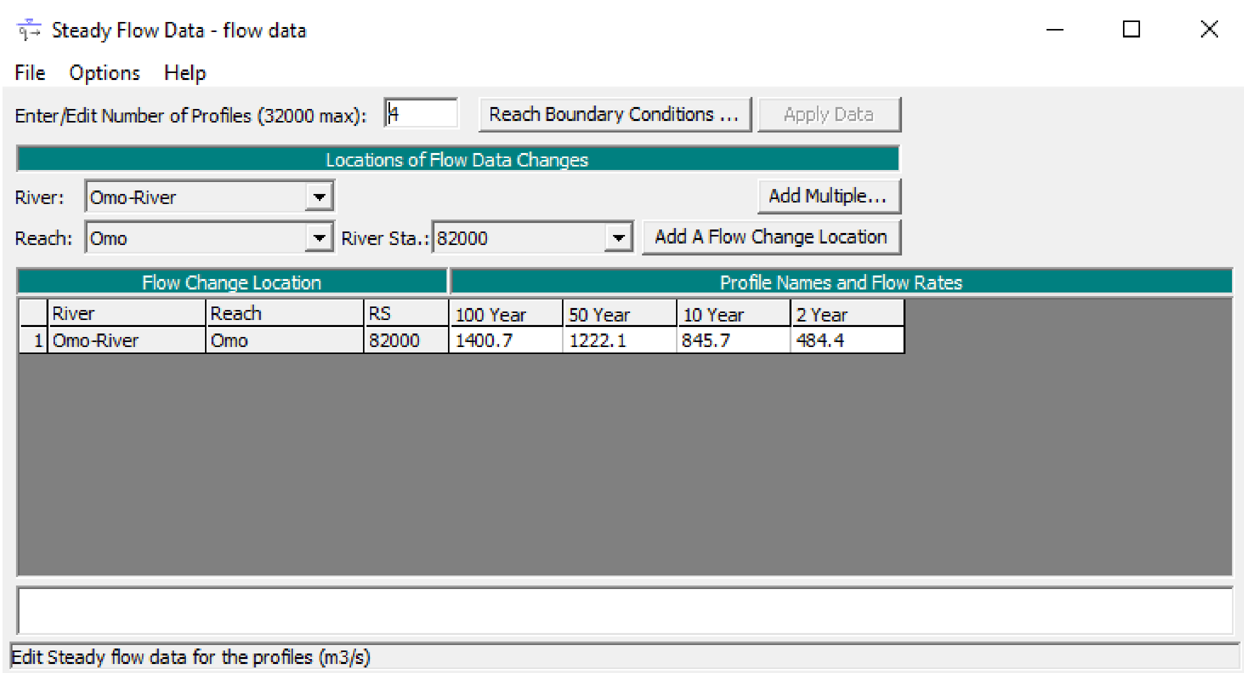


Figure 3-9 Steady Flow Condition of The Omo River Using Four Flow Profiles

3.13.3. PostRAS (HEC-GeoRAS)

With the development of a GIS export File from HEC-RAS, we can now begin the last portion of the model application. Post-processing using GeoRAS incorporates the water surface profiles derived from the HEC-RAS model into the spatial environment of GIS. The water surface profile data is used to develop a water surface TIN, and the intersection of the water surface TIN with the terrain model TIN provides flood visualization. The results can be shown in 2-D or 3-D views.

3.13.4. Roughness Values

Manning roughness coefficient used to represent the resistance to flood flows in channels and flood plains. Selection of an appropriate value for Manning's n values is very important to compute water surface profiles. Many factors can affect roughness, such as: Surface roughness, Vegetation, Channel irregularities, Channel alignment, Scour and deposition, Obstructions, Size and shape of the channel, Stage and discharge, Seasonal changes, Temperature and Suspended material & bed load etc. Selection of the proper value of the coefficient of friction, n, is very significant to the accuracy of the computed profiles. Manning equation can be solved for n when discharges corresponding to observed water-surface profiles are known. If no records are available le, values of n computed for similar stream conditions or values obtained from experimental data should be used as guides in selecting n values.

Tables and photographs for selecting n values provided in hydraulics text books, such as Chow (1959), may be used. Chow (1988) has presented Cowan's procedure of estimating the value of initially a smooth, straight and uniform natural channel is assumed and a value number is assigned. Different values are added to no considering surface irregularities, variation in shape and size of channel cross-sections, channel obstructions, vegetation cover and flow conditions and meandering of the river to get a final n representing the channel. Cowan's formula is given as: -

$$n = (n_0+n_1+n_2+n_3+n_4)m_5 \ldots \ldots (3.14)$$

Proper values of n_o to n_4 and m_5 are selected from Appendix B and a value of 0.055 has been obtained using the above equation and fixed for Omo River, as given on Table 3.7.

Table 3-7 Computation of manning coefficient n for Omo river

Channel Condition		Values	
Material Involved	Earth	no	0.02
Degree of Irregularity	Minor	n1	0.005
Variations of channel cross section	Alternating Occasionally	n2	0.005
Relative effect of obstruction	Negligible	n3	0.00
Vegetation	Higher	n4	0.025
Degree of Meandering	minor	m5	1.000

3.13.5. HEC-RAS Calibration

HEC-RAS, Automate roughness with AHYDRA Software for Sensitivity analysis. Select your RAS project Enter River and Reach name and also enter the manning for channel, banks and range of cross-sections to change 2D plot,3D plot and results in table.

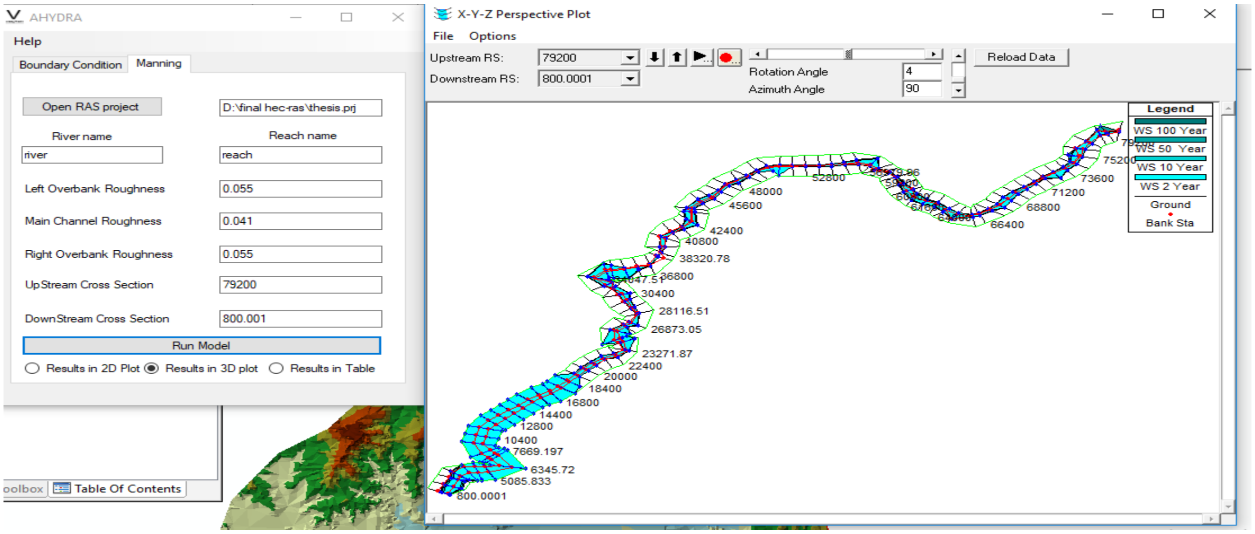


Figure 3-10 X-Y-Z perspective plot (results in 3D Plot)

3.14. 2D (Two-Dimensional) Flow Modeling Using HEC-RAS 5.0.1

2D flood mapping is carried out using the new version HEC-RAS 5.0.1 RAS mapper this help to compare and identify important information after analysis.

1D models contain mathematical simplifications related to the assumption that flood depth rests uniform over the entire cross-section. This assumption is not accurate for wide and flat floodplain areas.

2D models, use full dynamic or simplified forms of one- and two-dimensional shallow water equations to solve both one-dimensional channel flow and two-dimensional overland flow and are more appropriate for flat and wide floodplain areas. Because of this to identify the velocity of flood in meander of river in thesis include 2D Simulation and inundation mapping. So, in this study 2D flood simulation help to see every point cell information in RAS mapper.

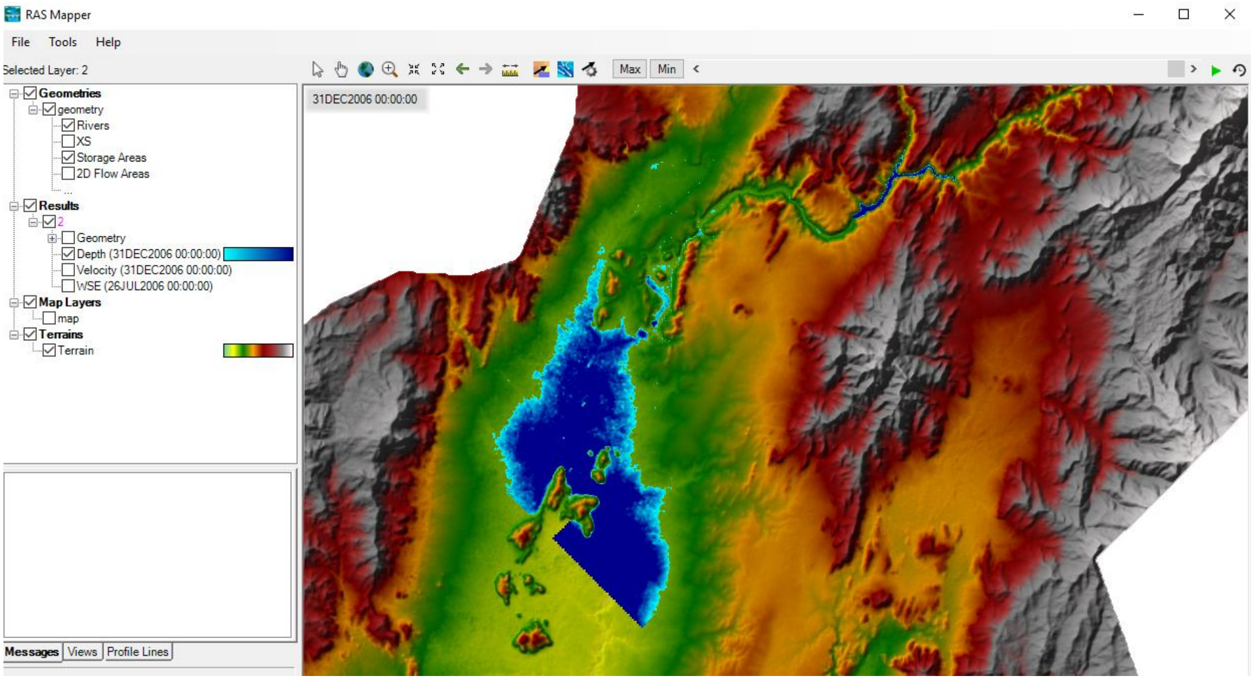


Figure 3-11 2D Simulation of flood depth in RAS mapper of the Omo Kuraz district

CHAPTER FOUR

4. RESULTS AND DISCUSSIONS

4.1. Homogeneity of the stations

Before using any rainfall data, would have to check the homogeneity for the analysis of the areal precipitation.

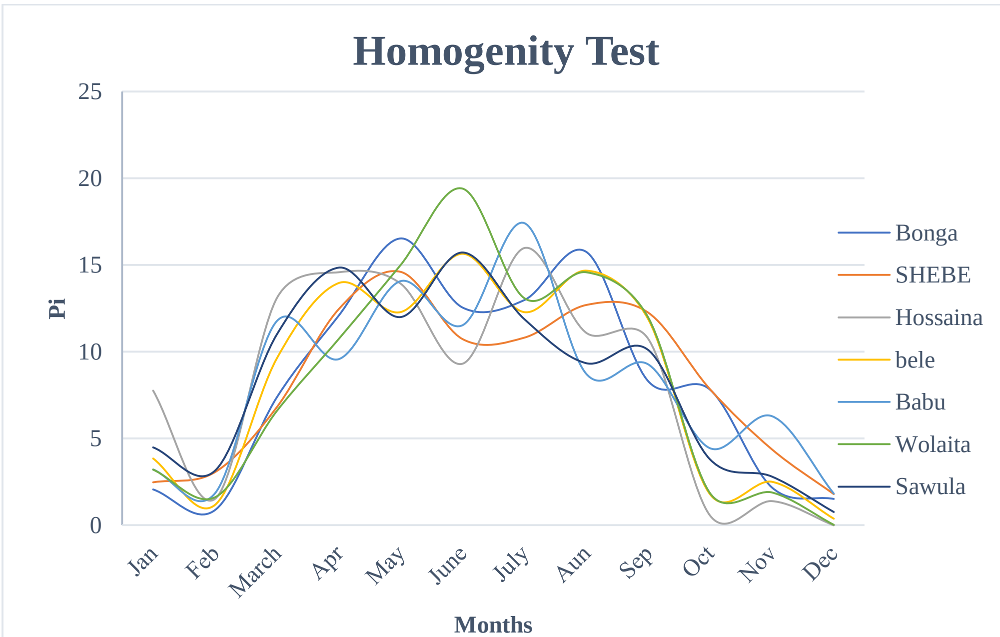


Figure 4-1 Homogeneity Test for rainfall stations

4.2. Consistency of recording stations of rainfall data

The trend of the rainfall records at a station may slightly change after some years due to a change in the environment (or exposure) of a station either due to coming of a new building, fence, planting of trees or cutting of forest nearby which affect the catch of the gauge due to change in the wind pattern or exposure. If the conditions relevant to the recording of a rain gauge station have undergone a significant change during the period of record, inconsistency would arise in the rainfall data of that station. This inconsistency would be felt from the time the significant change took place. The checking for inconsistency of a record was done by double mass curve analysis technique (Subramanya, 1998). The accumulated totals of the gauge in question are compared with the corresponding totals for a representative group of nearby gauges. If a decided change in the regime of the curve was observed it should be corrected. However, as all the selected stations in this study were consistent, there is no need of further correction.

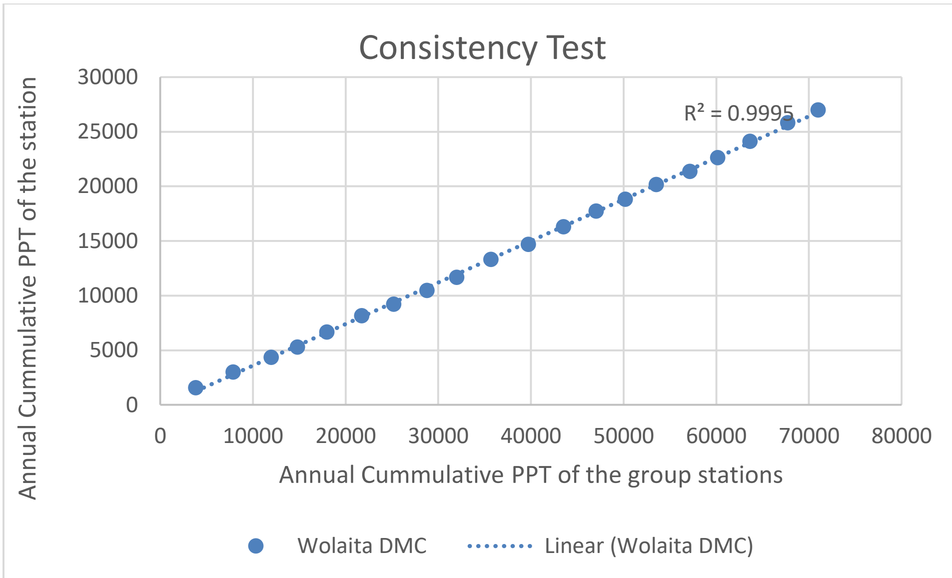


Figure 4-2 Double Mass plot of Wolaita Vs Hossaina, Bele and Sawula

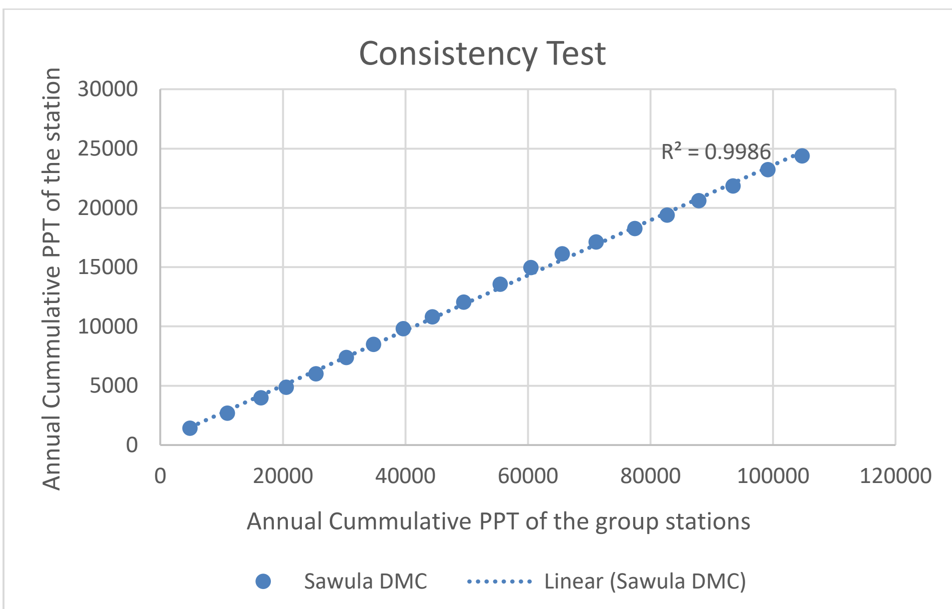


Figure 4-3 Double Mass plot of Sawula Vs Bonga, Shebe and Babu

4.3. Determination of Areal Rainfall

A rain gauge records the rainfall at a single point. This point rainfall record has to be converted to aerial rainfall. The average depth of precipitation over the area under the area of consideration was one of the most important parameters in hydrological analysis (AlHallaq, 2008).

The computation of average areal model input data may be done by the following methods:

1. Arithmetic average method: -When the rainfall is uniformly distributed over the area, the average rainfall may be taken as the arithmetic average of the recorded rainfall.
2. Thiessen polygon method: - Rainfall varies in intensity and duration from place to place. Hence the rainfall recorded by each rain gauge station should be weighted according to the area it is assumed to represent.
3. Isohytal method: - isohyets are a line joining places of equal rainfall intensities on a rainfall map of the basin. An isohytal map represents an accurate picture of the rainfall distribution over the basin. If the network rainfall stations within the storm area are sufficiently dense, the isohytal map were given a reasonably accurate indication of the rainfall distribution zones.

For catchments having rainfall gauges more than one, Theisen polygon method has been used to compute the areal rainfall.

The seven locations of the rain fall stations are shown in Figure 4.4.

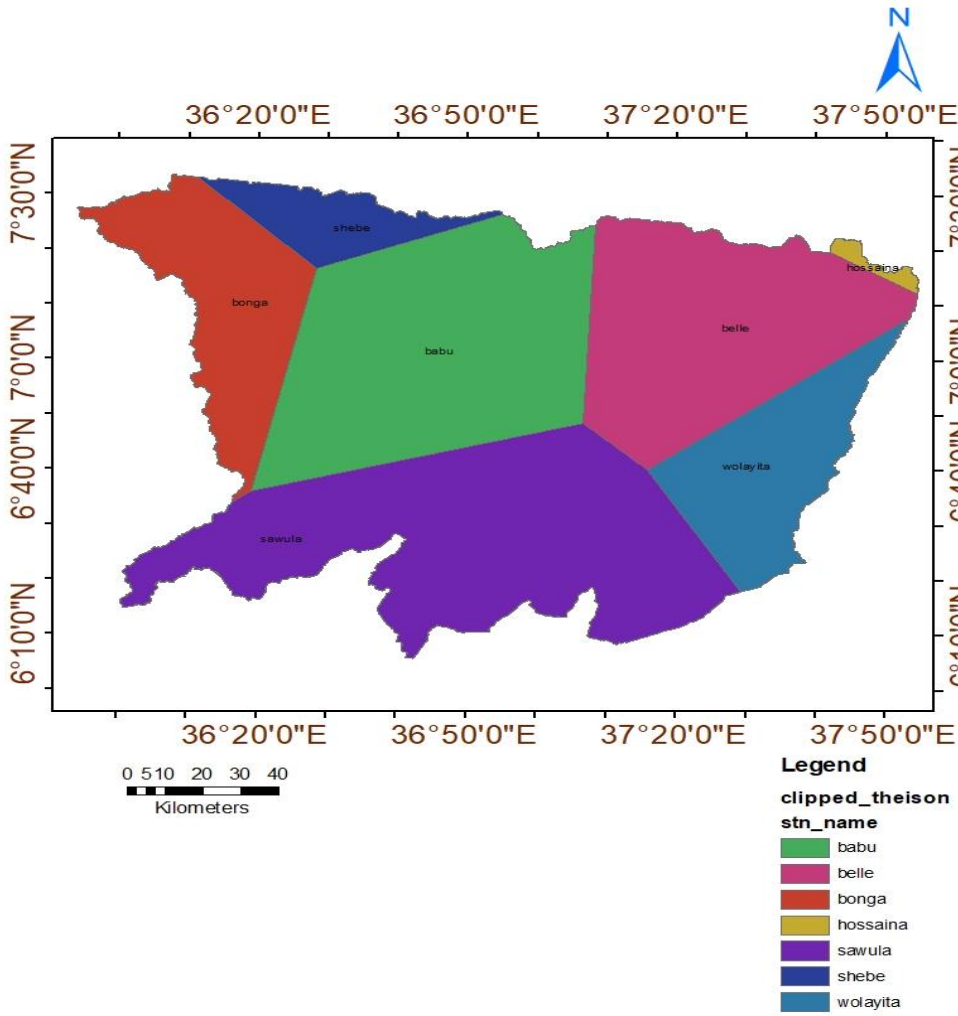
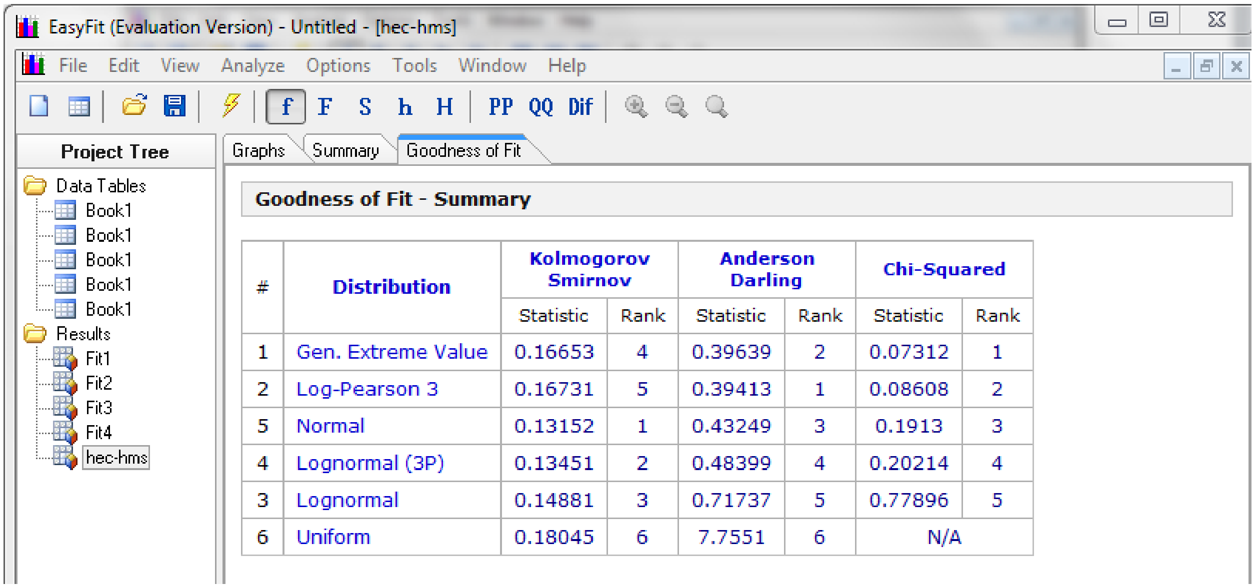


Figure 4-4 Clipped Theisen polygon of lower Omo Gibe River basin

4.4. Best fit flood probability distribution

The test statistics for Kolmogorov-Smirnov test (D), Anderson-Darling Test (A^2) and Chi-square test (x^2) for yearly discharge data were computed for six probability distributions. The probability distribution having the first rank along with their test statistic is presented by Easy fit software in Table 4-1.

Table 4-1 Goodness of Fit (GOF) Statistics Comparison

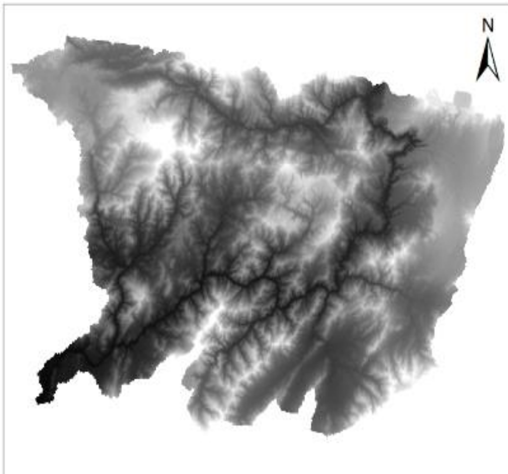


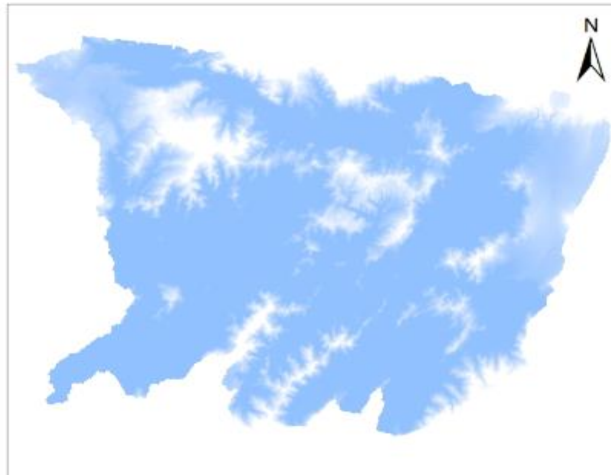
#	Distribution	Kolmogorov Smirnov		Anderson Darling		Chi-Squared	
		Statistic	Rank	Statistic	Rank	Statistic	Rank
1	Gen. Extreme Value	0.16653	4	0.39639	2	0.07312	1
2	Log-Pearson 3	0.16731	5	0.39413	1	0.08608	2
5	Normal	0.13152	1	0.43249	3	0.1913	3
4	Lognormal (3P)	0.13451	2	0.48399	4	0.20214	4
3	Lognormal	0.14881	3	0.71737	5	0.77896	5
6	Uniform	0.18045	6	7.7551	6	N/A	

We compare the process of fitting for six kinds of distribution. Since Goodness-of-Fit statistics are in form of distance between data and fitted distributions, clearly the distribution with minimum statistics value has been best fitted with data. Based on this fact, Easy Fit were attribute a ranking number to each distribution.

Using Kolmogorov-Smirnov test (D) it was observed that Normal distribution provides good fit to the yearly discharge data at the outlet. Using Anderson-Darling Test (A^2) it was observed that Log-Pearson type Three. Finally using Chi-squared test (x^2) has been applied to test the Goodness–of-Fit it has been shown that the generalized extreme value distribution provides good fit at the outlet. Comparing three goodness-of-fit tests it has been observed that the Generalized Extreme Value distribution provides a good fit for selected discharge at the outlet. Additionally, distributions are sorted based on results from Chi-squared test statistics, and the best fitted distribution (Generalized Extreme Value) is shown at top of the list.

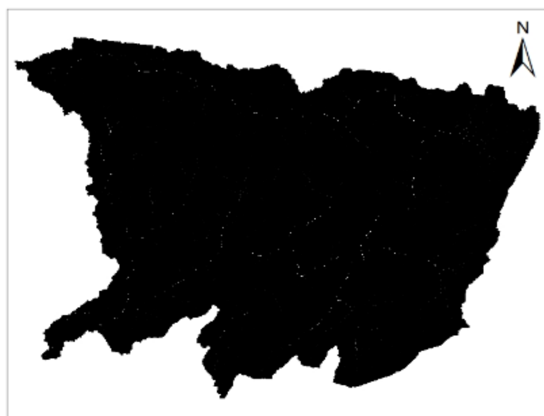
4.5. Terrain processing using arc hydro and HEC-GeoHMS





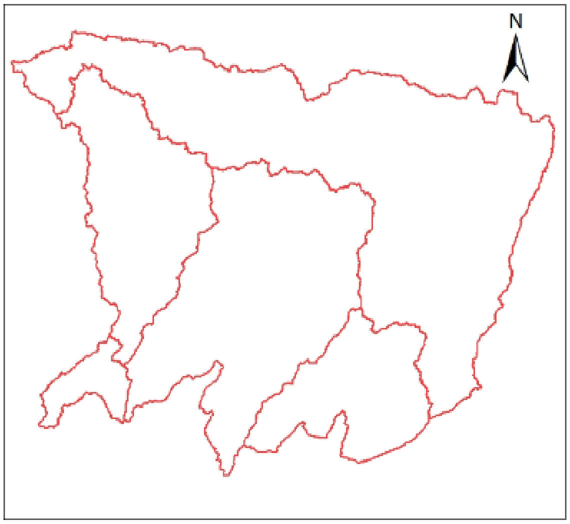
Raw dem

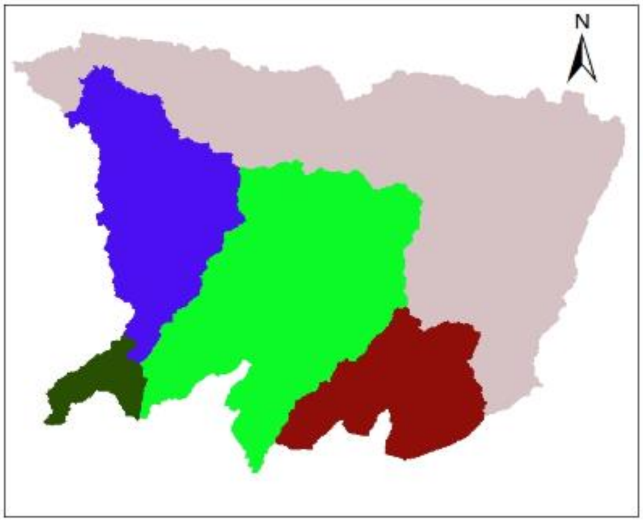
fill sink



Flow direction

flow accumulation





Catchment grid delineation

Catchment polygon processing

Figure 4-5 Terrain processing for Lower Omo Gibe catchment using the arc-hydro

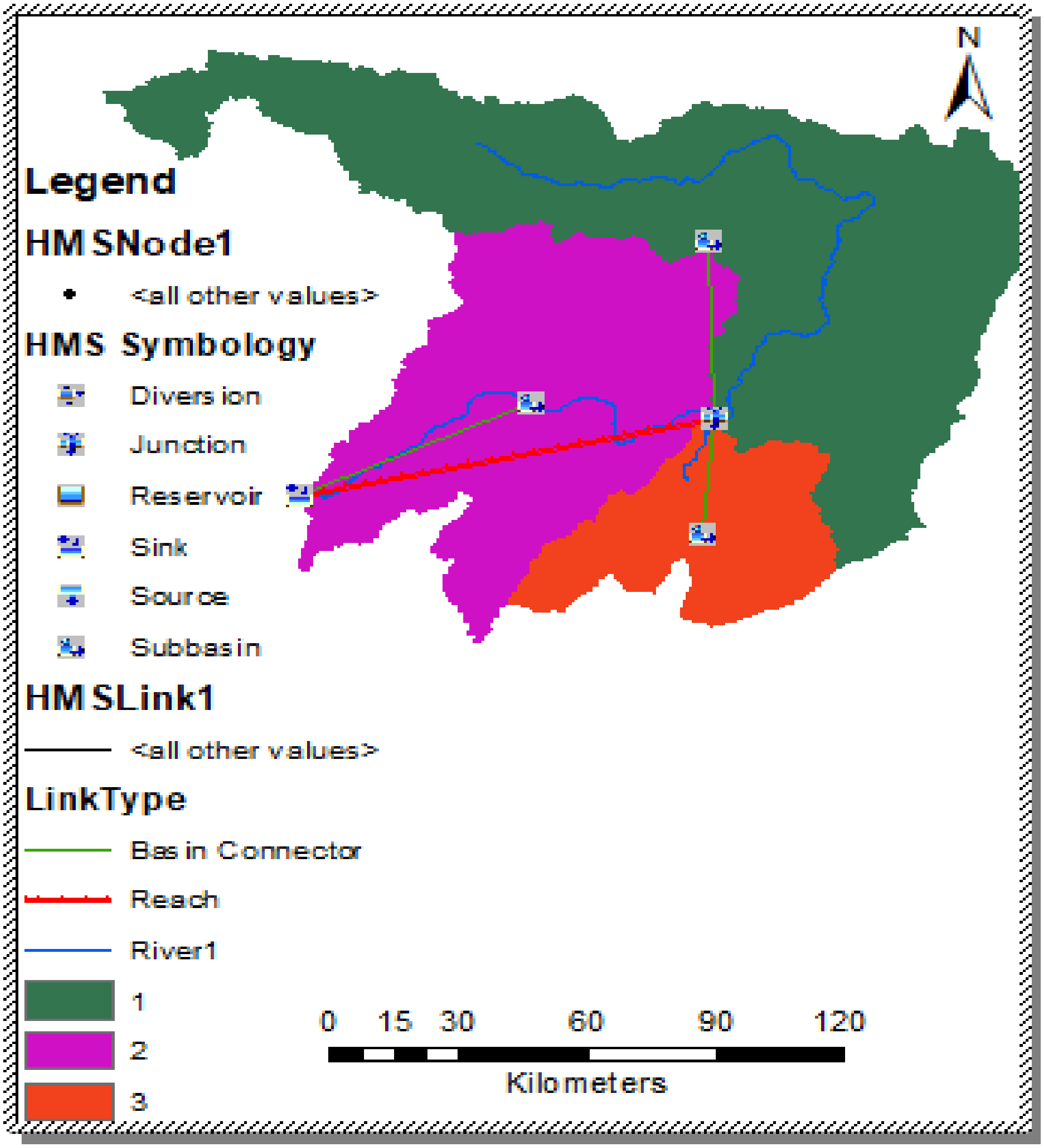


Figure 4-6 HMS Legend and Schematic using the HEC-GeoHMS

The output from terrain processing in Arc Hydro is not only delineation and schematic for the catchment but also extraction of basin characteristics from physical properties of the catchment. Among the basin characteristics soil and land use are the major ones. According to the output of the model the following parameters are generated.

Table 4-2 Catchment characteristic parameters extracted with Arc Hydro and HEC-GeoHMS

Component	Parameter	Unit	Value
	CN	Dimensionless	73.34
	IA	mm	18.47
	Im	%	0.0
Sub-basin 1	A	Km^2	10369.64
	CN	Dimensionless	70.47
	IA	mm	21.29
	Im	%	0.0
Sub-basin 2	A	Km^2	6109.24
	CN	Dimensionless	72.98
	IA	mm	18.81
	Im	%	0.0
Sub-basin 3	A	Km^2	2416.53
Sub-basin 1	Lag time	Min	1227.8
Sub-basin 2	Lag time	Min	771.48
Sub-basin 3	Lag time	Min	435.24

Applied HEC-HMS model to determine the peak discharge and volume of run-off they used SCS curve number method and SCS unit hydrograph to determine rainfall losses and watershed hydrograph respectively. Model parameters showed that the amount of CN is highly sensitive, while the initial abstraction was less sensitive to changes in the objective function in HEC-HMS model.

4.5.1. Creating CN Grid

HEC-GeoHMS was used to create the curve number grid. HEC-GeoHMS uses the merged feature class and the lookup table (CNLookUp) to create the curve number grid.

Data Requirement and description for curve number grid as shown below:

i) Clipped DEM for the study area (30x30metre DEM) Source: ASTER

ii) Clipped soil shape file of study area which consists of the Hydrological Soil Group of Ethiopia. Source: MWIE

iii) Extracted land cover shape file for the whole catchment from Ethio-GIS and current land use map of the basin. Source: MWIE

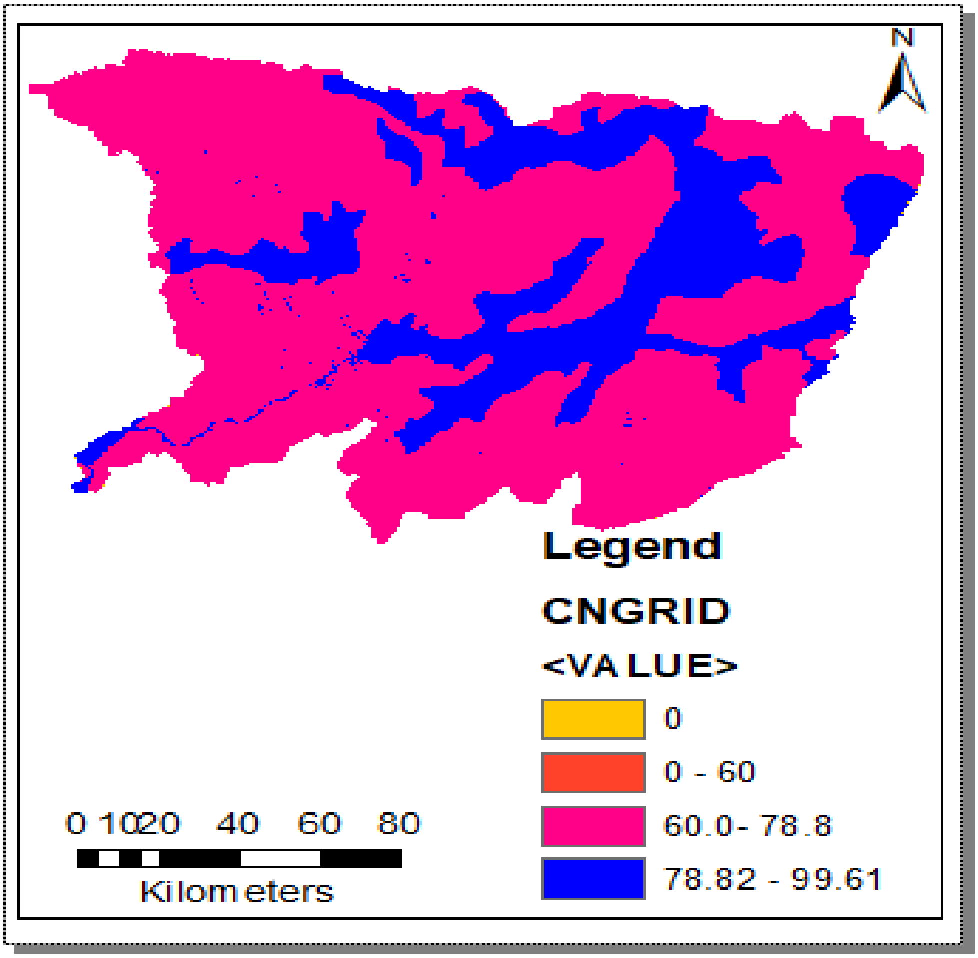


Figure 4-7 Curve Number Grid of the Catchments

4.6. HEC-HMS

4.6.1. HEC-HMS Calibration

HEC-HMS calibration was performed for a period of ten years (1998 to 2007) on the Omo Gibe Watershed of the area using daily flows basis. As discussed previously in chapter 3 the flow was calibrated automatically and manually using the observed flow at the outlet of Omo-Gibe watershed. Optimization of the parameter values was carried out within the allowable ranges recommended by the US Army corps of Engineers Hydrologic Engineering Center (Technical Reference Manual l March 2000).

The model results as obtained from the final automatic calibration using the peak weighted root mean square error objective function showed that there was a good agreement between the simulated and observed Omo Gibe catchments. This was demonstrated by the correlation coefficient and the Nash-Sutcliffe (1970) efficiency values for catchments. The daily calibration results are presented in Table 4.3. The statistical index of modeling efficiency(ENS)values range from 1.0(best) to negative infinity. ENS is a more tough test of performance than R^2 and is never larger than R^2.ENS measures how well the simulated results predict the measured data. A value of 0.0 for ENS means that the model predictions are just as accurate as using the measured data average to predict the measured data. ENS values less than 0.0 indicate the measured data average is a better predictor of the measured data than the model predictions while a value greater than 0.0 indicates the model is a better predictor of the measured data than the measured data average.

The Value of each parameter found in HEC-HMS must be specified to use the model for estimating run-off volume and routing hydrographs. Some of the model parameters cannot be estimated by observation or measurement of the watershed characteristics. For example, the parameter X and K in the Muskingum routing model cannot be measured but can be estimated approximately.

Optimization begins from initial parameter estimates and adjusts them so that the simulated results match the observed stream flow as closely as possible.

Table 4-3 Optimized Parameters of HEC-HMS for Omo Gibe Catchment, Daily Basis

Optimized Parameter Results for Trial "Trial 1"

Project:Middle   Optimization Trial:Trial 1

Start of Trial: 01Jan1998, 00:00   Basin Model: Middle
End of Trial: 31Dec2007, 00:00   Meteorologic Model:Middle
Compute Time:04Aug2018, 17:19:18

Element	Parameter	Units	Initial Value	Optimized Value	Objective Function Sensitivity
W40	SCS Curve Number - Curve Number		67	35.456	0.05
W50	SCS Curve Number - Curve Number		70.470	38.054	0.03
W60	SCS Curve Number - Curve Number		59	35.223	0.01
W40	SCS Curve Number - Initial Abstraction	MM	90.55	85.225	0.00
W50	SCS Curve Number - Initial Abstraction	MM	58.77	55.314	0.00
W60	SCS Curve Number - Initial Abstraction	MM	290	500.00	-0.02
W40	SCS Unit Hydrograph - Lag Time	MIN	1427.8	1447.2	-0.00
W50	SCS Unit Hydrograph - Lag Time	MIN	1100.48	1035.8	-0.00
W60	SCS Unit Hydrograph - Lag Time	MIN	2600	3937.5	-0.00
R30	Muskingum - K	HR	145.55	147.68	-0.02
R30	Muskingum - x		0.39	0.36706	0.00

Algorithm included in the program search for the model parameters that yield the best value of an index, also known as objective function. Out of objective functions in HMS, Peak-weighted mean square error was selected for this study. This function is an implicit measure of comparison of the magnitudes of the peaks, volumes and times of peak of the two hydrographs. Univariate Gradient and Nelder Mead Algorithm method of search parameter that minimizes the value of the objective function. The objective function results Omo-Gibe catchment are presented in Table 4.4. The simulation is event based in all cases.

Table 4-4 Objective Function Results

Objective Function Results for Trial "Trial 1"

Project:Middle Optimization Trial:Trial 1

Start of Trial: 01Jan1998, 00:00 Basin Model: Middle
End of Trial: 31Dec2007, 00:00 Meteorologic Model:Middle
Compute Time:04Aug2018, 17:19:18

Objective Function at Basin Element "Outlet1"

Start of Function:01Jan1998, 00:00 Type: Peak-Weighted RMS Error
End of Function: 31Dec2007, 00:00 Value:174.00

Volume Units: (•) MM ( ) 1000 M3

Measure	Simulated	Observed	Difference	Percent Difference
Volume (MM)	5769.58	3871.55	1898.03	49.02
Peak Flow (M3/S)	909.0	1108.1	-199.1	-18.0
Time of Peak	01Sep2006, 00:00	21Aug2006, 00:00		
Time of Center of Mass	10Apr2003, 21:11	19Jan2003, 10:16		

Summary Results for Sink "Outlet1"

Project: Middle Optimization Trial: Trial 1
Sink: Outlet1

Start of Trial: 01Jan1998, 00:00 Basin Model: Middle
End of Trial: 31Dec2007, 00:00 Meteorologic Model:Middle
Compute Time:04Aug2018, 17:19:18

Volume Units: (•) MM ( ) 1000 M3

Computed Results

Peak Discharge:909.0 (M3/S) Date/Time of Peak Discharge01Sep2006, 00:00
Volume: 5769.58 (MM)

Observed Hydrograph at Gage ddd

Peak Discharge: 1108.1 (M3/S) Date/Time of Peak Discharge:21Aug2006, 00:00
Mean Abs Error:142.7 (M3/S) RMS Error: 165.2 (M3/S)
Volume: 3871.55 (MM) Volume Residual: 1898.03 (MM)
Nash-Sutcliffe: 0.505

Figure 4-8 Summary Results for sink: outlet

Flow hydrographs for the observed and simulated flows at Outlet gaging station is presented in the graph below

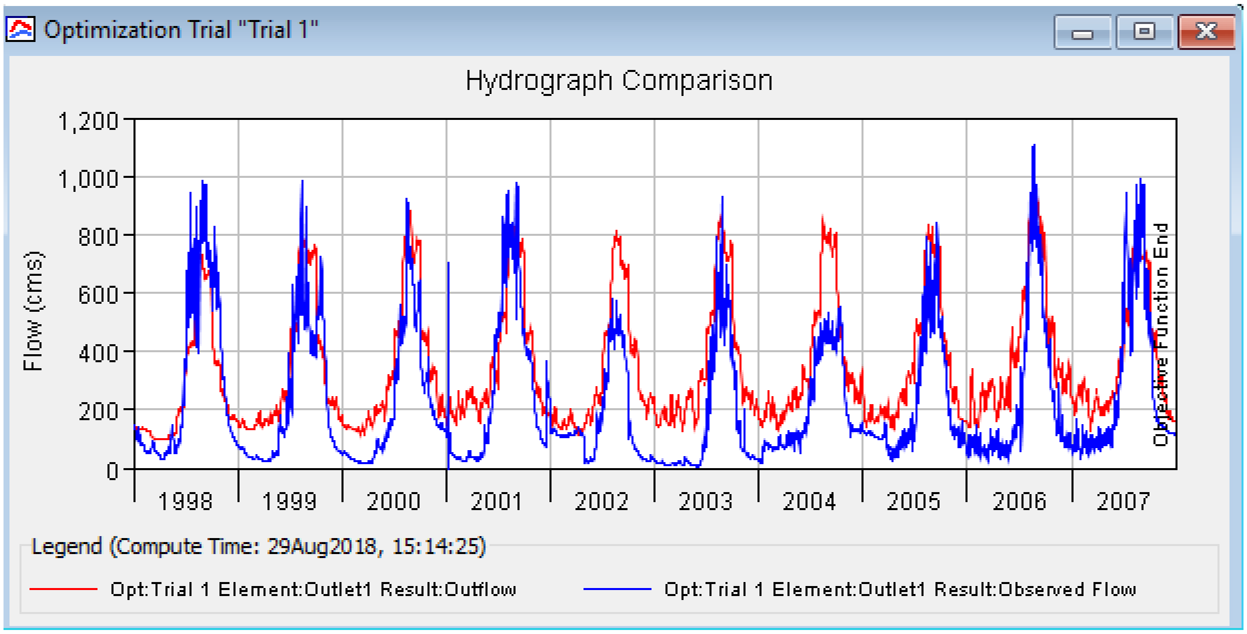


Figure 4-9 Daily Hydrograph Comparison Between Resulted HEC- HMS Out Flow versus Observed Flow

Maximum discharge and minimum discharge of both the observed and simulated discharge almost similar to each other, but for the year 2002 and 2003 the simulated data over estimate the observed data.

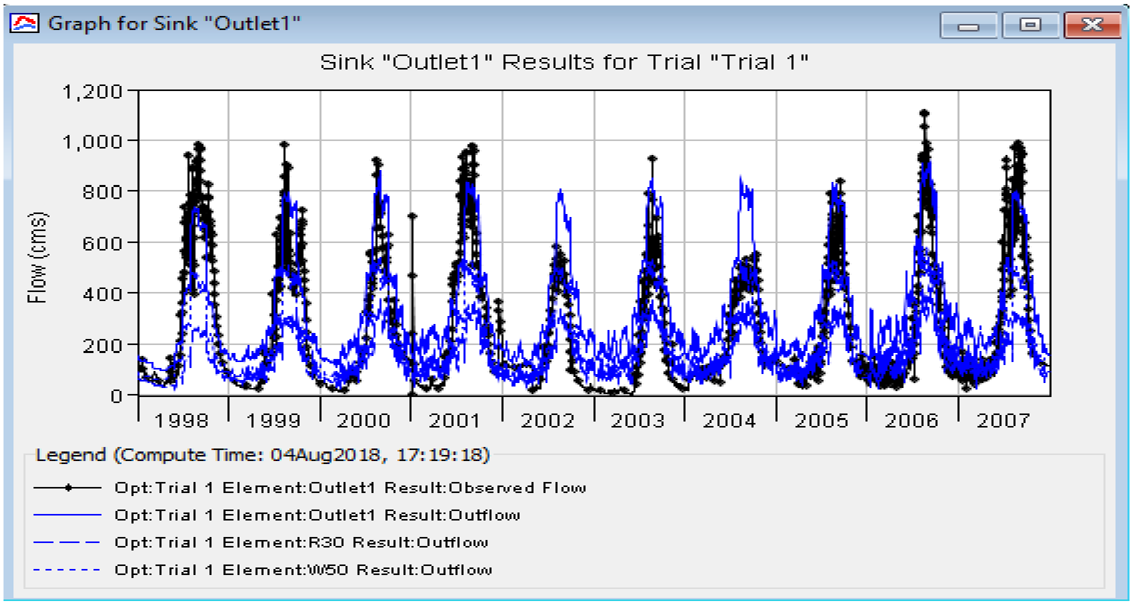


Figure 4-10 Calibration of HEC-HMS Observed and Simulated Daily Flow Hydrographs

The results of calibration and validation of the model is presented in the Table 4.5.

Table 4-5 Calibration Performance of HEC-HMS for Lower Omo Gibe Catchment

performance	Lower-Omo Gibe
Coefficient of Determination(R^2)	0.737
Nash-Sutcliffe (E_{ns})	0.505
Period of Calibration	From 1998 to 2007

Both the Calibration and Validation results showed good match between measured and Simulated stream flow and rainfall-data with acceptable range, coefficient of determination(R^2) (0.68 to 0.88 For Calibration and 0.62 to 0.86 for Validation and Nash-Sutcliffe efficiency(ENS) (0.5 to 0.88 for Calibration and 0.61 to 0.85 for Validation.

Table 4-6 Calibration Performance of HEC-HMS for Lower Omo Gibe Catchment

Performance	Lower-Omo Gibe
Coefficient of Determination(R^2)	0.758
Nash-Sutcliffe (E_{ns})	0.61
Period of Validation	From 2008 to 2015

Both efficiency evaluation criteria parameters almost resemble each other. For the Omo Gibe case the E_{ns} has a value of 0.505 and R^2=0.737 for calibration and E_{ns} has a value 0.61 and R^2=0.758 for validation and also the observed discharge overestimates the simulated discharge.

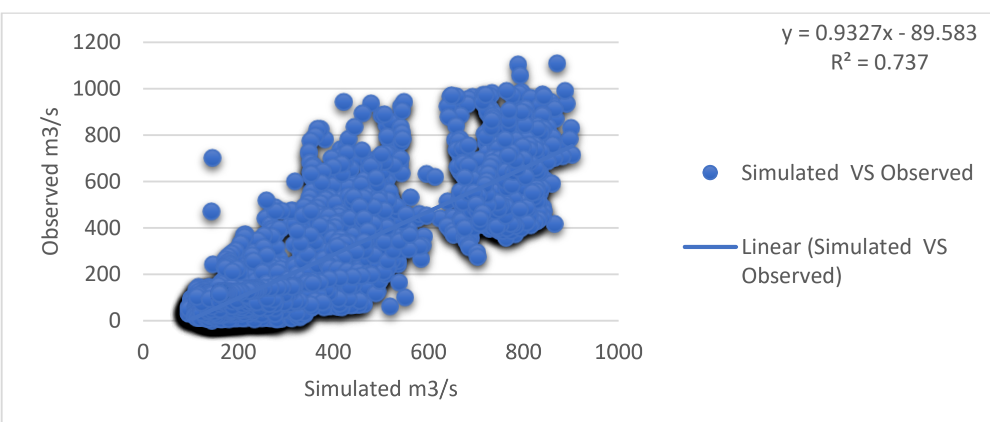


Figure 4-11 Correlation between simulated versus observed at the Outlet

The model performance is validated accordingly. For the HEC-HMS model there are a total of 20 years taken for both calibration and validation. The validation time was selected at the last eight years of the total years.

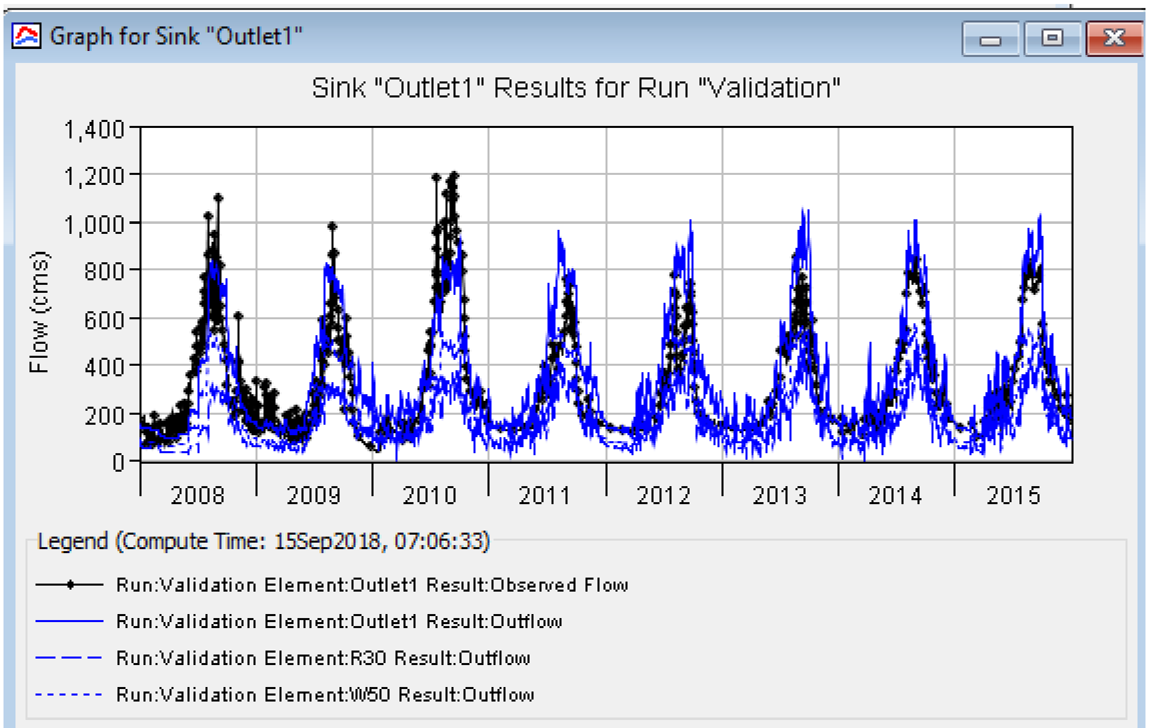


Figure 4-12 Validation of HEC-HMS Observed and Simulated Daily Flow Hydrographs

The parameter imported from the HEC-GeoHMS where include SCS Curve number method (curve number, initial abstraction and impervious), SCS unit Hydrograph method (Lag time, Graph type) and Muskingum model (X and K Coefficient).

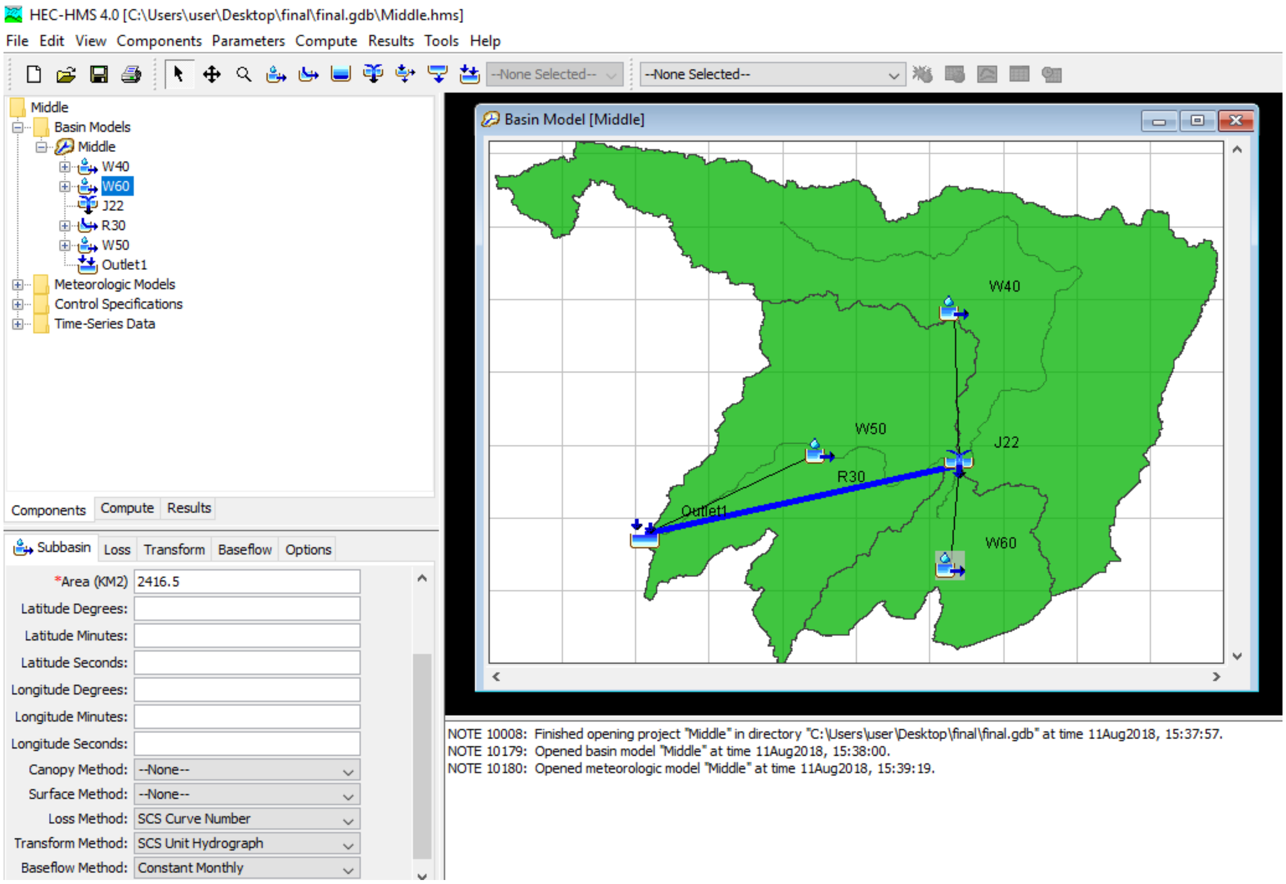


Figure 4-13 HEC-HMS schematic imported from HEC-GeoHMS

4.6.2. Output of HEC-HMS by Frequency Storm

Using the parameters obtained from the daily basis the model results peak flows for the following return periods 2,10,50 and 100 years and the flow values are found accordingly.

Table 4-7 Determination of Peak Discharge Using HEC-HMS Frequency Method

S.NO	Return Period (Years)	Peak Flow(m^3/s)
1	2	484.4
2	10	845.7
3	50	1222.1
4	100	1400.7

From the result table minimum peak flow for the Omo Gibe River is occurred for 2-year return period for 24-hour storm duration and the maximum obtained with 100-year frequency storm for the same duration. The value being 484.4 m^3/s and 1400.7m^3/s for 2 year and 100-year frequency respectively.

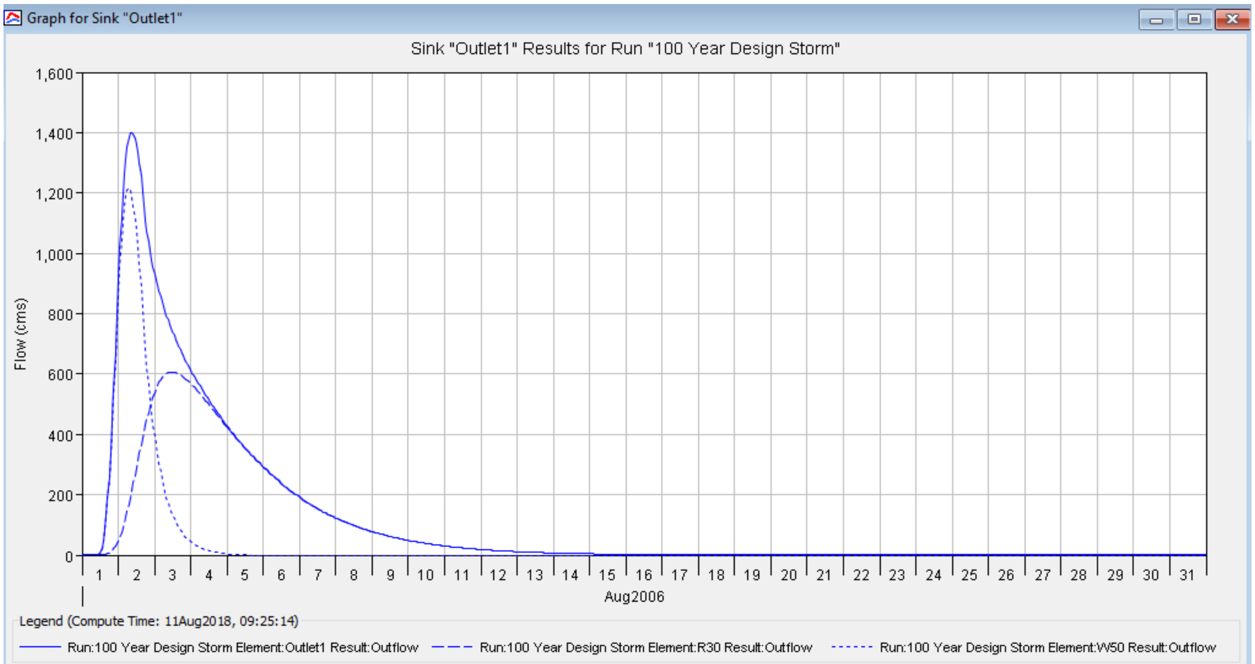


Figure 4-14 100-year HEC-HMS Frequency storm Flow of Omo –Gibe at Outlet

The HEC-HMS model result was compared with the frequency analysis results. they are selected using software called Easy Fit Software for selection of methods. According to the output the following four popular methods are selected.

Table 4-8 Flow value Comparison (frequency analysis and the HEC-HMS)

Method	$Q_t(m^3/s)$				
		2	10	50	100
Frequency Analysis	Gen. Extreme Value	830.89	1091.84	1320.51	1417.3
	Normal	165.45	1322.81	2020.32	2266.47
	Lognormal	891.25	1583.45	2238.89	2529.99
	Log-Pearson Type III	921.57	1095.71	1159.61	1175.37
HEC-HMS		484.4	845.7	1222.1	1400.7

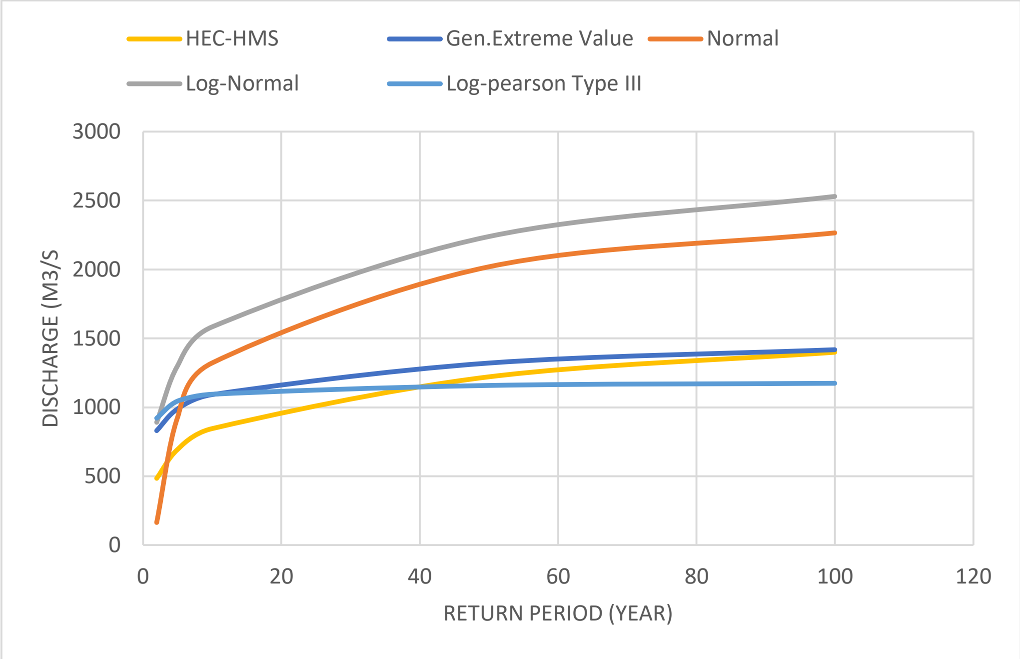


Figure 4-15 Comparison of the flow values at the outlet of HEC-HMS

In the above table 4-8 and figure 4-15 the frequency discharge value derived using General Extreme value method show high similarity to the HEC-HMS. The other two are higher than the result of the HEC-HMS and also one distribution was lower or higher than the value of the HEC-HMS.

4.7. Pre RAS-processing using HEC-GeoRAS

The goal of this part was to develop the basic spatial data required to generate the HEC-RAS Geometry Import File. The process required is the Generation of a digital terrain model (in this paper TIN is generated from the DEM of the study area), Definition of base 2D spatial features and Generation of 3D spatial data and HEC-RAS Geometry Import File. With the DTM/TIN generated earlier, the next step is 2D spatial feature definition.

4.7.1. 2D Spatial Features Definition

With the digital terrain representation (TIN) created, the next step is to extract the geometric information required by HEC-RAS. This step started with the delineation of a series of 2D spatial features corresponding to the stream centerlines, the left and right bank lines, the flow paths, and the cross sections along the streams. The contour lines may be helpful in this regard if the resolution of the TIN is poor. In general, the delineation of cross sections located close to river junctions was not easy: each cross-section had to cross the stream centerline exactly once, the bank lines exactly twice (left and right), and the flow paths exactly three times (left, right and centerline) and they should not intersect each other.

4.7.2. Cross section geometry

Cross sections should be perpendicular to the anticipated flow lines and extend across the entire flood plain (these cross sections may be curved or bent). The cross section for this research work is extracted from digitized TIN. The study area TIN is made from the DEM of the area. Before digitizing the cross section, the stream layers must be made available. The layers are; stream center line, flow path center line, flow path lines (left and right) and bank lines.

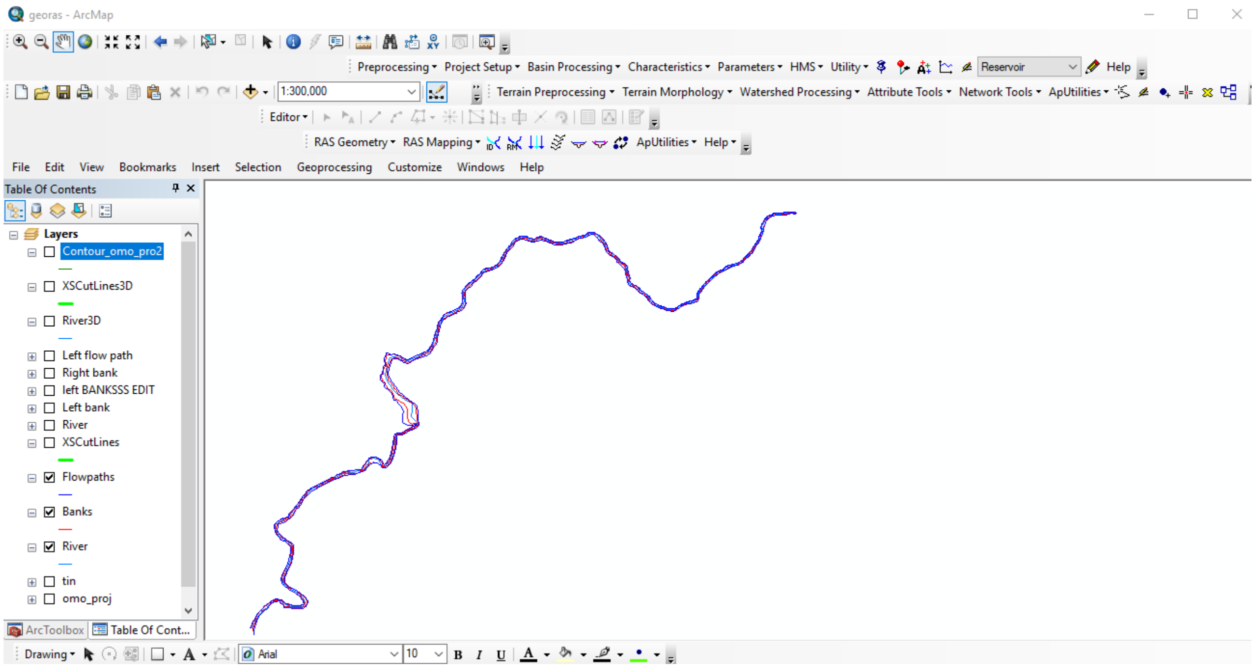


Figure 4-16 A digitized Omo River with RAS layers around the Omo Kuraz district

The above five features are extracted from prepared TIN of the study area. The TIN was generated from DEM of the area. During digitizing shape files of the river or contour can be used to follow.

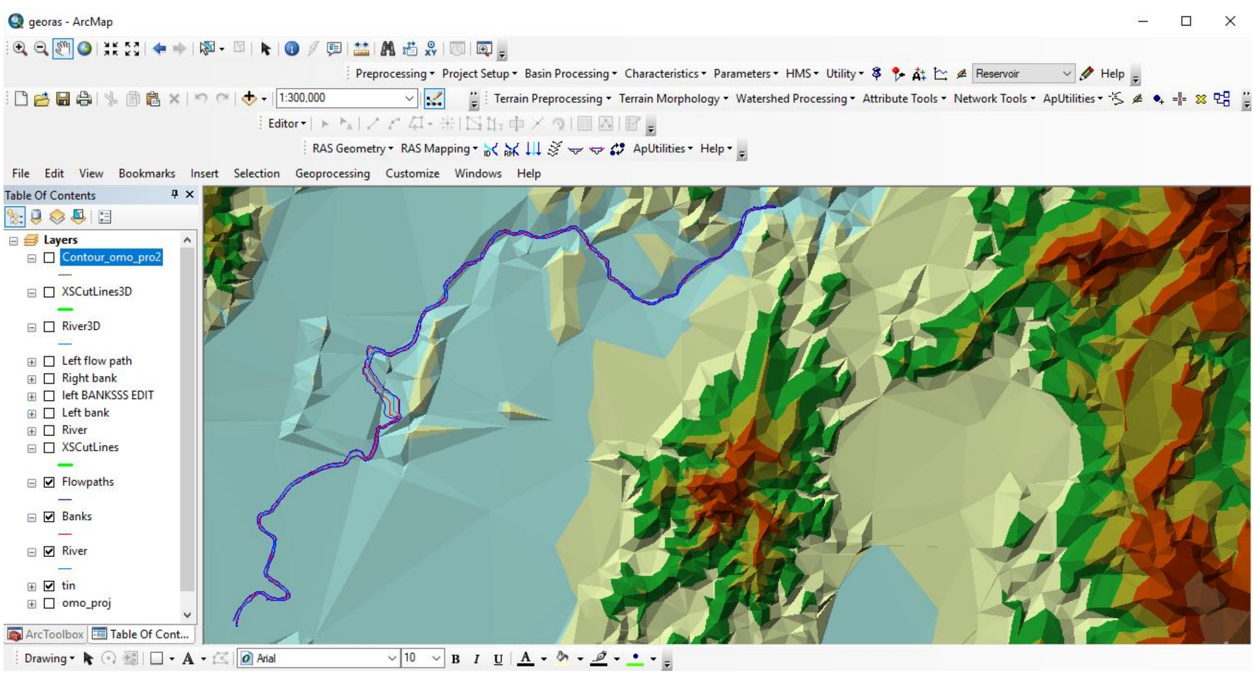


Figure 4-17 An underlying TIN and associated lower- Omo Gibe River

Preprocessing by HEC-GeoRAS in ArcGIS is the first step in the extraction processes. The step is used in geo-referencing and digitizing the stream layers for further use in HEC-RAS.

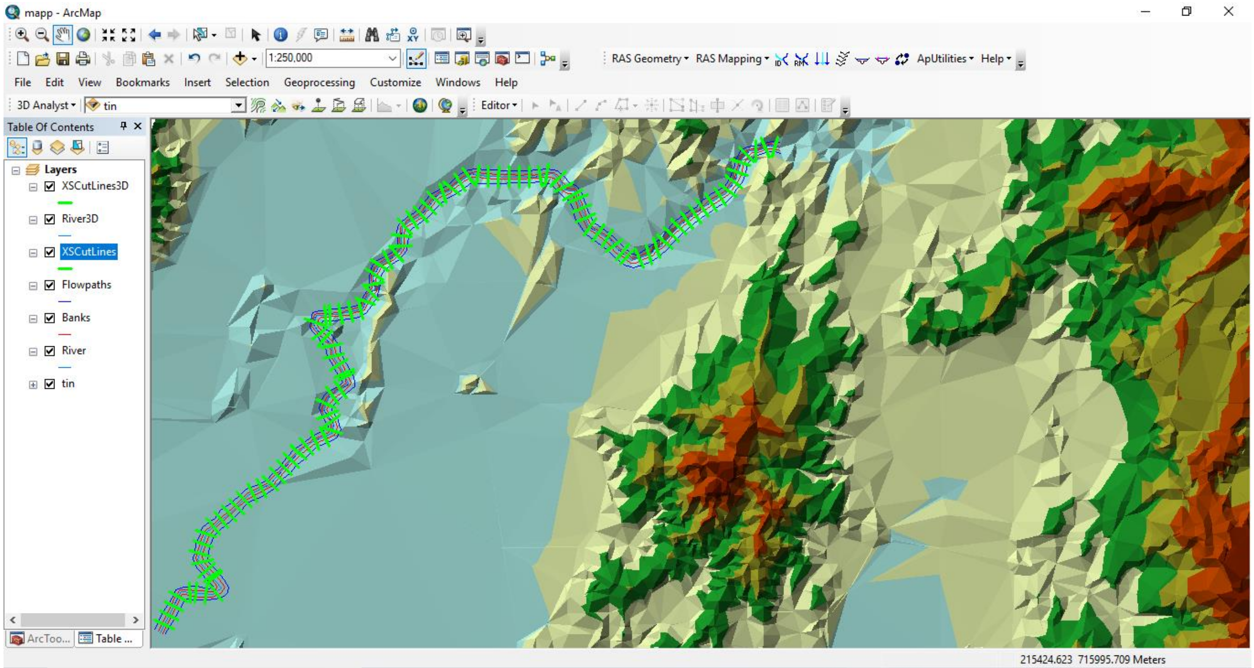


Figure 4-18 Cross section of the lower-Omo Gibe River digitized in GIS10.3

4.7.3. 3D Spatial Features and HEC-RAS Geometry Import File Generation

The 3D spatial data generation involved creation of 3D stream centerlines and 3D cross-sections, with *Z* values to define elevations. The *Z* values were extracted from the TIN. Once generated, the 3D features identified the stream network and the HEC-RAS model layout. The generated cross section is then changed to polyline *Z*. The 2D point data is changed to the 3D polyline due to the extraction of elevation from the DTM/TIN.

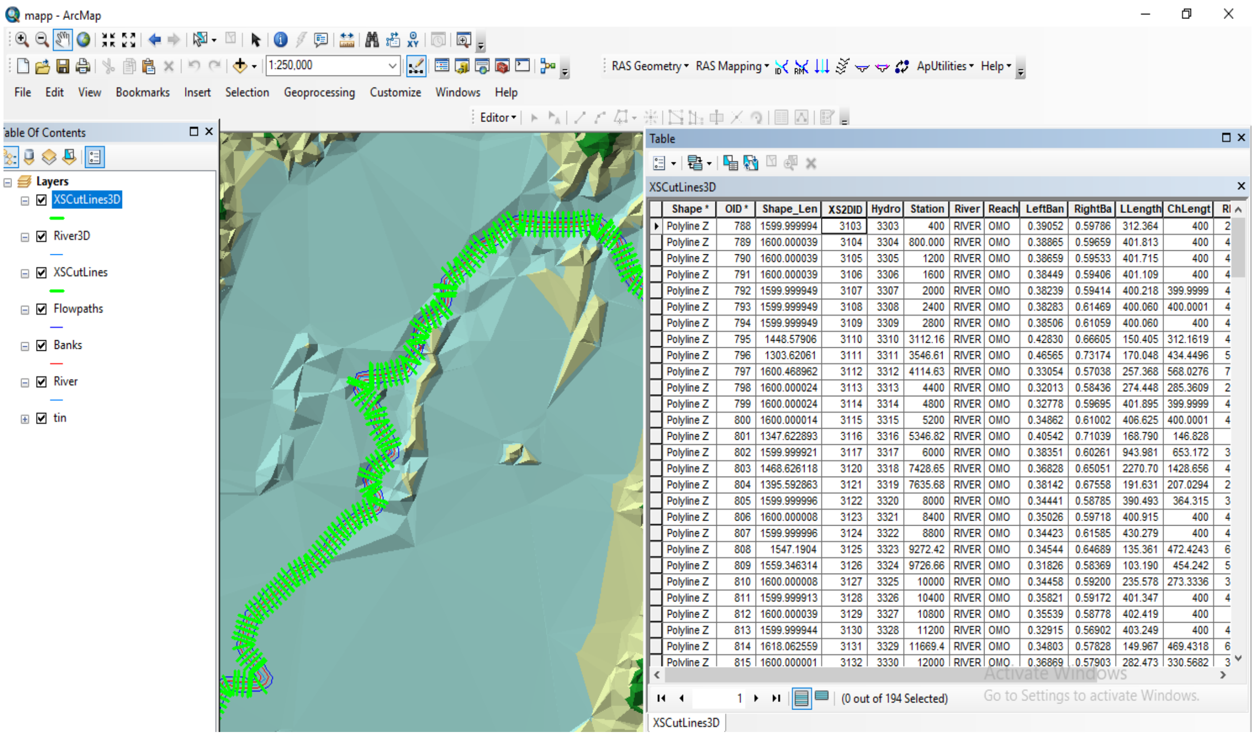


Figure 4-19 3D spatial feature generated after extraction of elevation from TIN

4.7.4. Exporting to HEC-RAS

It is very important to edit and geo-reference all necessary layers in GIS. Although the HEC-RAS has an editing interface for the exported value, the GIS is a better way to reduce the error during post-RAS process (flood mapping and delineation). There are different options to leave or export RAS layers depending on their use and necessity. There may be errors during pre-RAS processes. The bank stations which are made fit with the cross-section points in GIS may not match when exported to HEC-RAS. In this case manual edition should be applied. The exported cross section may not also be readable by the HEC-RAS. The problem may emerge from the unit system between the HEC-RAS and that used in GIS. The GIS unit system must be re-projected according to the RAS unit. Since most GIS inputs such as DEM and TIN are in metric unit it must be projected to the same unit.

4.7.5. Importing and Editing Geometric Data

The first of the components is the channel geometry. Analyze stream flow, HEC-RAS represents a stream channel and floodplain as a series of cross-sections along the channel. Create our geometric model, we need to import the geometry file that just exported from ARCGIS. This HEC-RAS geometry file contains physical parameters describing cross-sections.

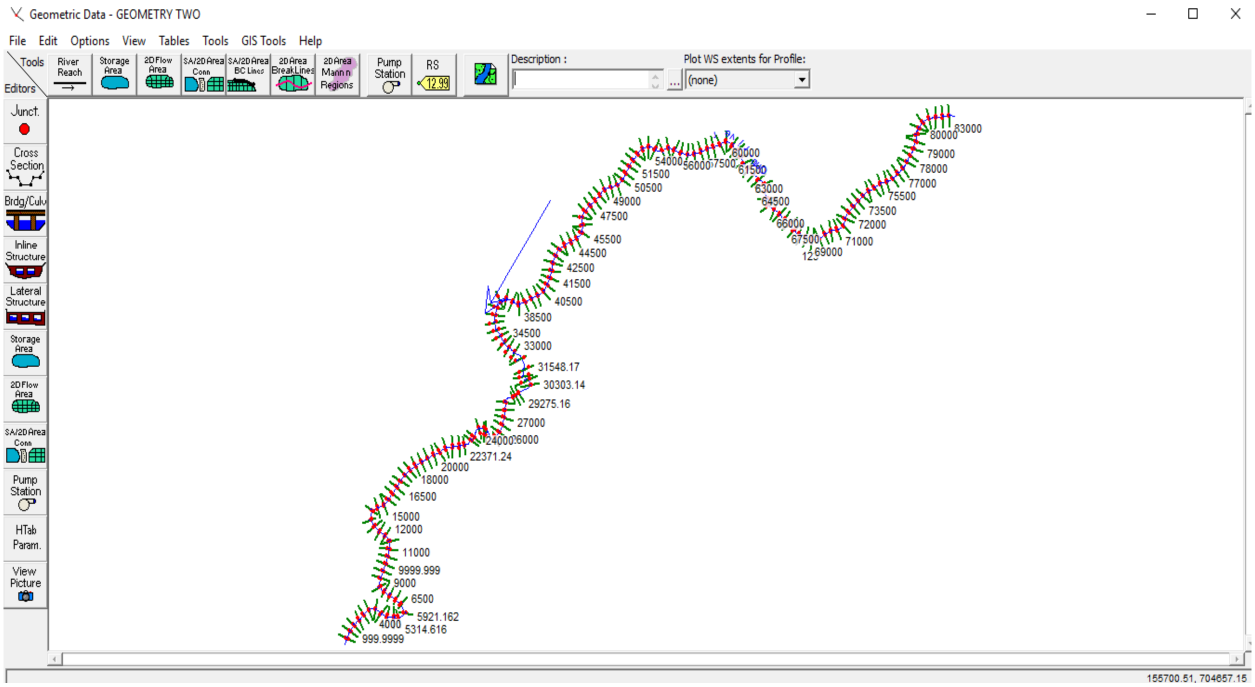


Figure 4-20 Geometry Data of the River Flood Plain

4.7.6. HEC-RAS Output

The model HEC RAS needs flood plain cross-section and flow data which is determined using the hourly data frequency method using HEC-HMS which is determined below. Inserting the flow data and importing the Cross-section data from ARC GIS using the HEC -GeoRAS post processing, by running the model for the four flow condition profiles to determine the water surface profile of the reach. The simulation is made for steady flow condition. Simulated water surface profiles for flood of 100-year return period at selected cross sections where there is over breaching of riverbank.

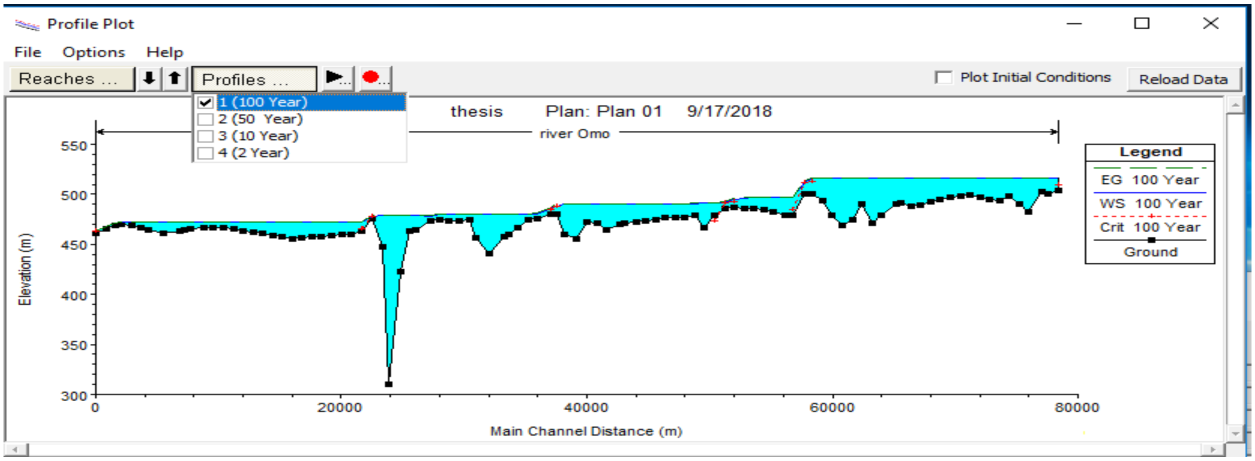


Figure 4-21 Water Surface Profile for 100 - year Return Periods around the Omo Kuraz district

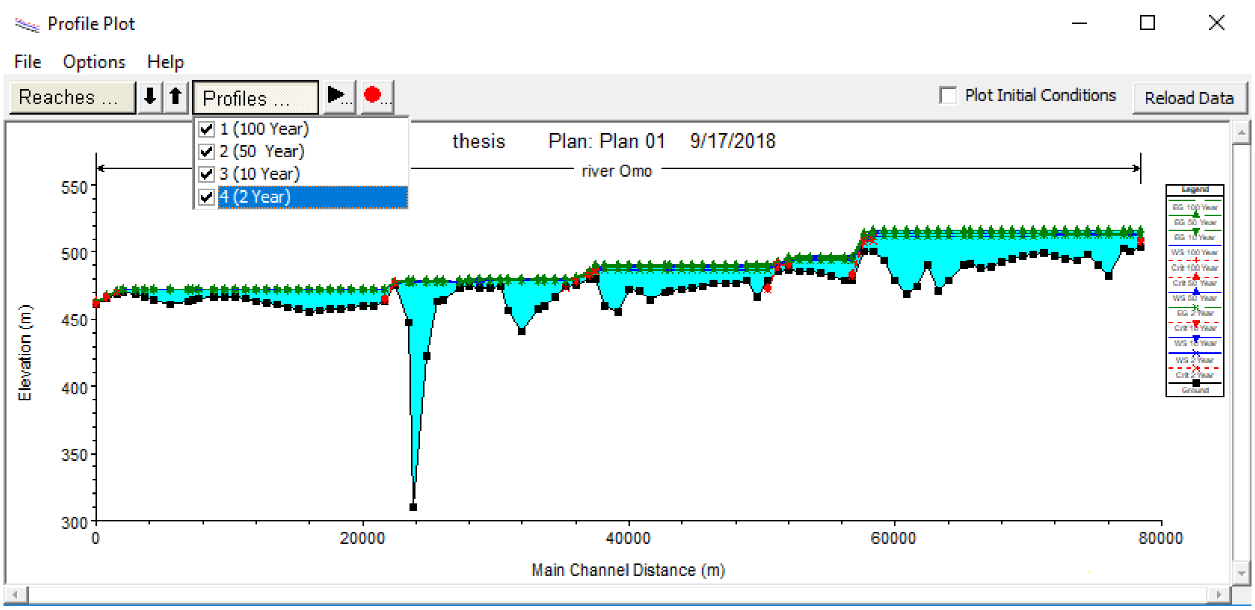


Figure 4-22 2D profile plot for all design period around the Omo Kuraz district

4.8. Mapping Flood Plains Using GIS

With the bounding polygon created, water surface TIN was created from the given profiles and underlying DTM/TIN. The water surface TIN consequently gives rise to flood plain delineation. The flood plain mapping was completed in two steps with generation of water surface TIN from cross section water surface elevations and water surface TIN is intersected with the digital terrain model to create flood plain polygons for flow scenario. Using the HEC-GeoRAS post processing exporting the HEC-RAS out puts into the Arc GIS the flood plain inundation mapping and delineation is done using the Arc-Gis tool bars.

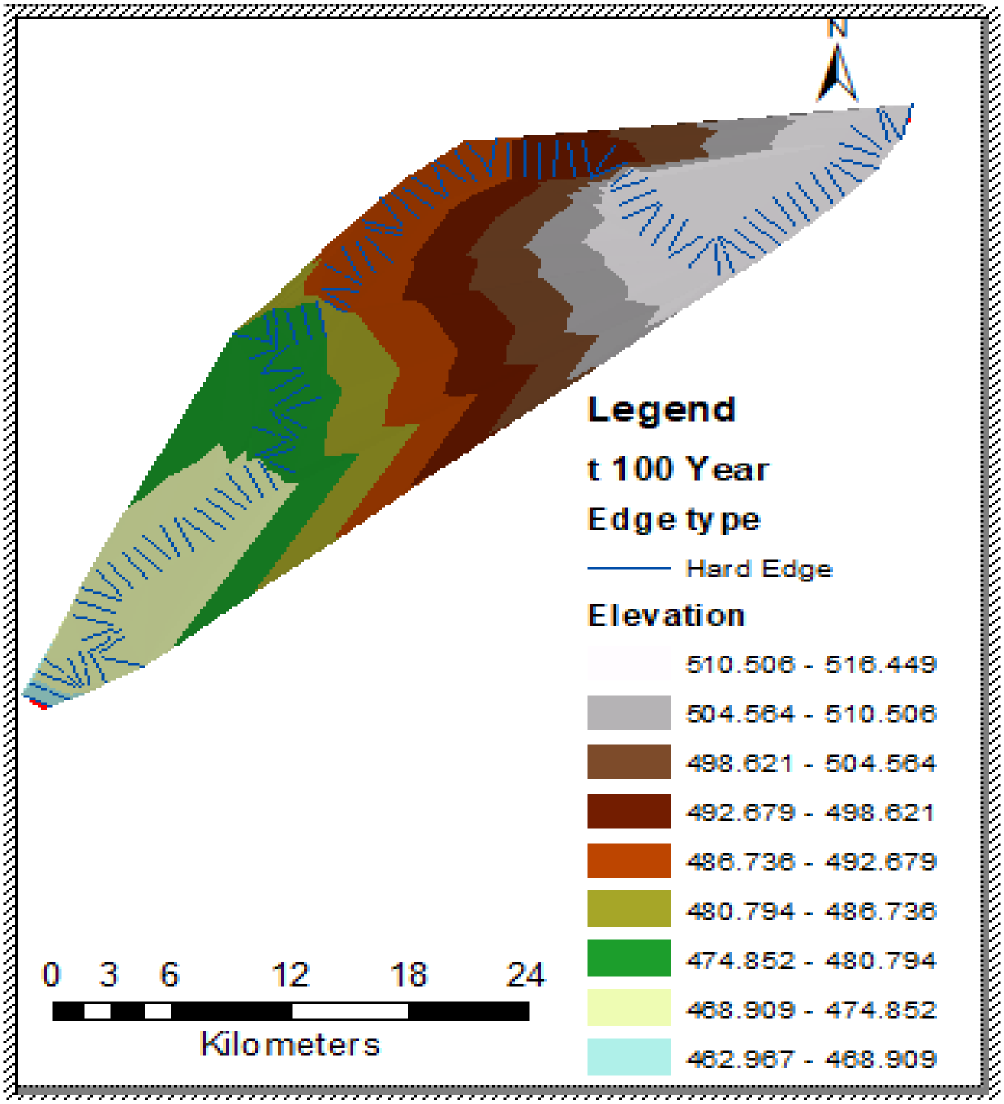


Figure 4-23 Water surface TIN generated from bounding polygon

ArcGIS with an extension of HEC-GeoRAS then delineates flood plain for different flow conditions. In this paper there are four storm flows considered (2,10, 50 and 100 year). Flood plain map for each differ in depth, extent and area. From the figure 4-24 for 100-year storm event, it can be seen that the depth of the flood ranges from 0m-9.494m.

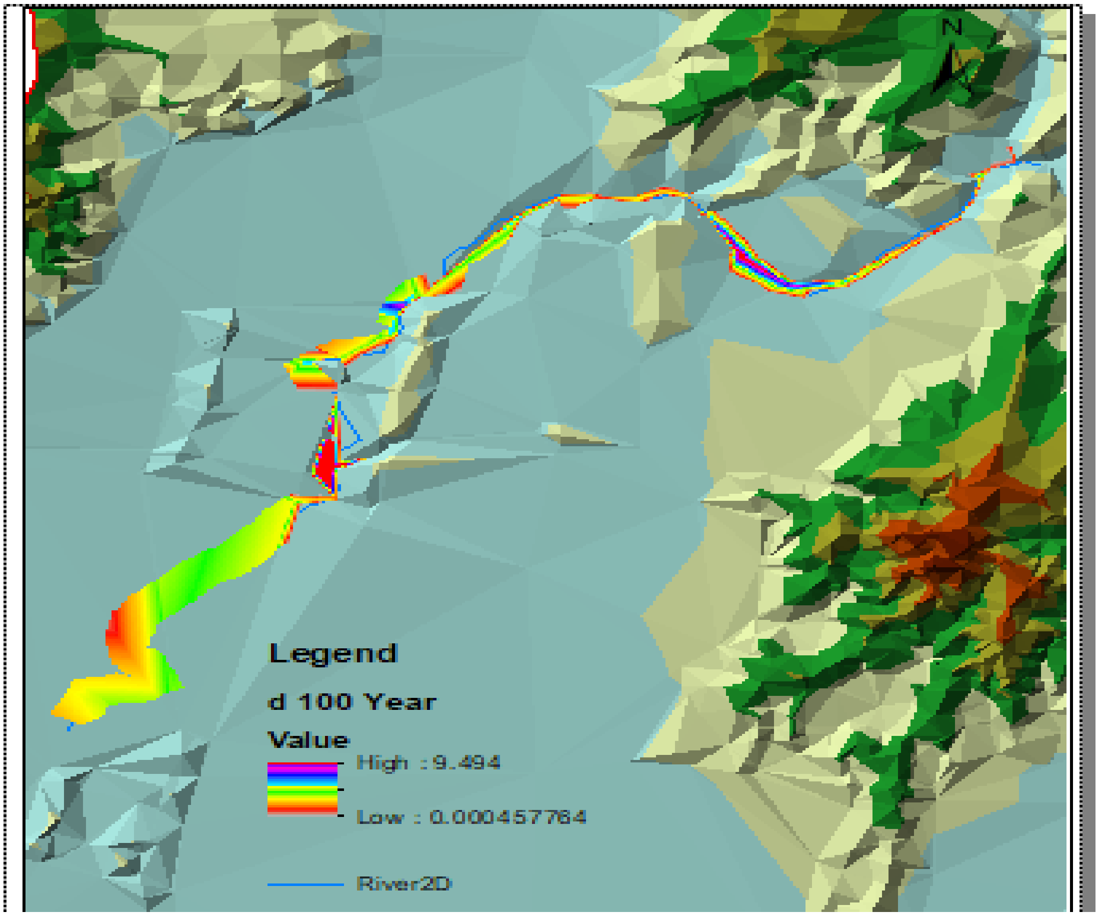


Figure 4-24 100-year flood map and depth for the study area

After the HEC-GeoRAS post processing was completed, the flooded area and shape length 76.94 km^2,169.14km respectively in the figure 4-24 for the 100-year flood frequency for mixed flow and also 44.951km2 and 98.6km from figure 4-25 for the 2-year flood frequency.

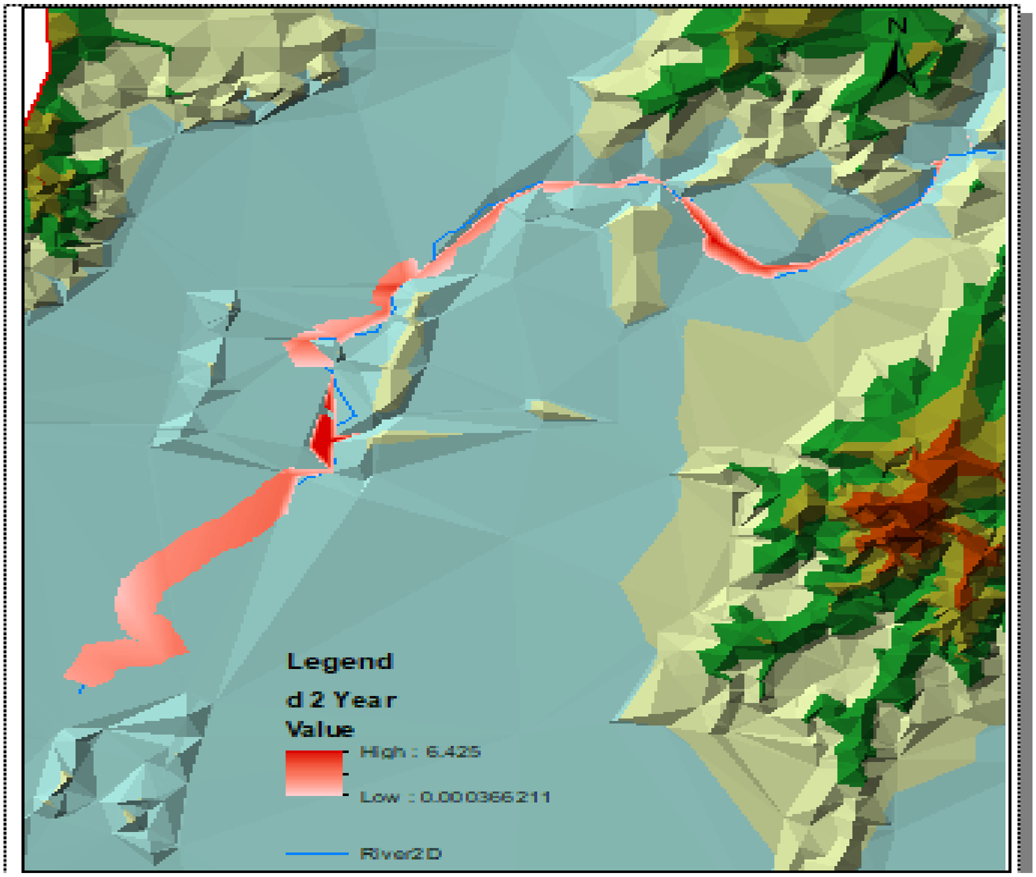


Figure 4-25 2-year flood map and depth for the study area

CHAPTER FIVE

5. CONCLUSION AND RECOMMENDATION

5.1. Conclusion

Goodness-of-Fit tests can be used to compare fitted distributions, select a model, and determine how good the distribution was fitted to the data. Easy Fit software generates reciprocal reports which facilitate achieving a general perspective over fitted distributions as well as evaluating the level of fit goodness for certain models at various significance levels.

Flooding around Omo-Gibe River causes a considerable damage to life and property. Large coverage of the area with cultivated land makes the problem hard. Past flood forecasting of the area lacks the use of modern software. This study presents a systematic approach in the preparation of flood map and simulation with the application Such as steady flow models, Unsteady flow models and Arc GIS 10.3. The tools/models used in this method was HEC-HMS, One-dimensional numerical model HEC-RAS and ArcGIS for spatial data processing and HEC-GeoRAS, HEC-GeoHMS for interfacing between the HEC-HMS, HEC-RAS and ArcGIS.

100 years return period peak flood discharge estimated using Computer Programs HEC-GeoHMS and HEC-HMS found to be 909.0 m^3/sec and ERA IDF curves was used for flood analysis in plain. As explained above in chapter four, the 100 years and 2years flood frequency values are used for the flood mapping.

Identifying the distribution amount of monthly or yearly maximum discharge data could have a wide range of applications in agriculture, hydrology for Design period, engineering design and climate research.

The total area affected by this flood is76.94km^2 and the area affected by the 2year flood inundation of 98.6km is 44.951 Km^2.

Finally, to select the type of mitigation measure it is recommended to undertake further flood risk analysis study instead of the traditional way of design to protect the area from flood.

5.2. Recommendation

This study was conducted under limited data availability. Therefore, the following recommendations are made for the further studies in the future.

- Flow data: There was no gauged flow data at the confluence of the river specially at the middle and downstream of the river, flow data of long time duration is also necessary for the calibration and validation of hydrologic model. Unavailability of hourly meteorological data should be addressed.
- Up-to-date DEM*:* Should be adapted for high accuracy in representation TIN of the study area. For future work, GIS Based Hydraulic Modeling using HEC-GeoRAS can be done if a high-resolution DEM is found for the area. High resolution DEM may can be prepared from 1-meter contour map.
- A digital elevation model of 30meter was used for the delineation of the basin, which has low resolution. So, DEM which has high resolution should be used in order to improve the result.
- At downstream of Omo River there was no gauged data to compare the magnitude of flood. It recommended that for better justification for flood occurrence it should be set the gauging station at downstream Omo river around Omo rate.
- No Survey cross section data because there is no enough budget given by the ERA, as well as the site I was select very far from Addis Ababa.
- Use of new technology to generate TIN: TINs obtained using new technologies such as LIDAR (Light Detection and Ranging), which improves the quality of the digital terrain representations are better if used for further study.

REFERENCE

Abushandi, E. (2013). Modelling Rainfall Run-off Relations Using HEC-HMS and IHACRES for a Single Rain Event in an Arid Region of Jordan. Water Resource Manage, 19.

Ajumuka, O. C. (2013). probability distribution models for flood prediction in Upper Benue River Basin-part II. Civil and Environmental Research, 14.

Akerman. (2002). HEC-GeoRAS An extension for support of HEC-RAS using Arc View-GIS. US Army corps of Engineers Hydrologic Engineering Center 609 second street Davis,CA 95616.

Alem, M. A. (2018). Best-fit probability distributions and return periods for maximum monthly rainfall in Bangladesh. Climate, 16.

AlHallaq, A. H. (2008). Determination of Mean Areal Rainfall in the Gaza strip using Geographic information system(GIS) Technique. University of Sharjah Journal of pure and Applied Scieces Volume 5.No.2, 23.

Allen et al. (1998). Pereira LS,Raes D,Crop evapotranspiration,Guidelines for computing crop water re-quirements. FAO Irrigation and Drainage paper, 56.

Arekhi, S. (2012). Run-off modeling by HEC-HMS model(Case study:Kan watershed,Iran). International journal of Agricuture and Crop sciences, 5.

ArlenD.Feldman. (2000). Hydrologic Modeling System HEC_HMS Technical Reference Manual. U.S.Army Corps of Engineers HEC.

Asadi, A. (2013). Performance Evaluation of the HEC-HMS Hydrologic model for lumped and semi-distributed storm flow simulation(Study Area:Delibajak Basin). American Journal of Engineering Research, 7.

Avery, S. (2010). Hydrological Impacts of Ethiopia's Omo Basin on Kenya's Lake Turkana Water Levels and Fisheries. Nairobi, Kenya: Final Report, African Development Bank Group.

Awlachew, S. (2000). Investigation of water Resources Aimed at Multi-Objective Development with Respect to Limited Data Situation:The Case of Abaya-Chamo Basin,phD.Thesis.

Brunner, G. (2010). HEC-RAS,River Analysis System Hydraulic Reference Manual. Washingeton DC.

Davis. (2009). HEC-GeoHMS Geospatial Hydrlogic modeling extension users manual. Institue for Water resources.

Diabi, I. (2016). Hydrological Modelling of wadi Ressoul watershed,Algeria,by HEC-HMS model. Journal of water and land development, 9.

ERA. (2013). ERA Drainage Design manual. Addis Ababa: Ethipian Roads Authority.

Feldman. (2000). Hydrologic modeling system HEC-HMS Technical Reference manual. Washington DC 20503: U.S.Army Corps of Enginneers HEC 609 Second St.Davis,CA95616-4687.

Gary W.Brunner, C.-H. (February,2016). HEC-RAS River Analysis System Users Manual Version 5.0. US Army Corps of Engineers Institute For Water Resources Hydrologic Engineering Center(HEC).

Hargreaves, G. (1975). Moisture availability and crop production. transactions of the American society of Agricutural Engineers ASAE, ASAE,18(5).

Knebl et al. (2005). M.R, Yang Z.L., Hutchison K., Maidment D. R., Regional Scale Flood modeling using NEXRAD Rainfall, GIS, and HEC-HMS/RAS: Accese study for the san Antonio River Basin Summer 2002 storm event. Juornal of Environmental management, 75 pp.

Lampros, V. (2009). Hydrological response to metereological drought indices in thessaly. Greece ,journal of hydrologic Engineering.

Mehrannia, H. (2014). Using Easy Fit Software for Goodness-of-Fit test and Data Generation. International Journal of Mathematical Archive-5(1),2014,118-124, 7.

Merwade, V. (2010). Tutorial on using Hec-GeoRAS with ArcGis 9.3. Purdue University.

OCHA UN office of the co-ordination of humanitarian affairs, p. o. (2006). Retrieved from http:// WWW.relief web.int.

Ohimain, E. I. (2014). Selective impacts of the 2012 water floods on the vegetation and wildlife of wilber force island,Nigeria.

Olofintoye. (2009). Best-fit probability distribution model for peak daily rainfall of selected cities in Nigeria. New york Science Journal, 12.

Pakgohar, A. (2014). Usig Easy Fit Softwere for Goodness-of-Fit Test and Data Generation. International Journal of Mathematical Archive-5(1), 7(118-124).

Penman, H. (1948). Natural evaporation from open water,bare soil and grass. proc.Royal Soc.Londom,193(1032):, 120-145.

Priestley, T. R. (1972). Taylor RJ,On the assessment of the Surface heat flux and evaporation using large-scale parameters. Monthly Weather Review, 81-92.

Research, C. a. (2013). Probability distribution models for flood prediction in Upper Benue:River Basin. Benin.

Richard Woodroofe . (1996). Omo Gibe River basin integrated development Master plan study Final report Vol.III and VI. Addis Ababa: Water resources surveys andinventories,Ministry of water resources.

Roshun, H. (2011). Employing GIS toRiver Hydraulic Analysis Using HEC-RAS model (Case Study Zaremrood River,Mazandaran Province).

Solaimini, K. (2012). Employing GIS to River Hydraulic Analysis using HEC-RAS model (Case study:Zaremrood River,Mazandaran province). 3 Th Symposium Climate change and Dendrochronology, 12.

Subramanya, K. (1998). Engineering Hydrology ,Tata Mc Graw-Hill second Edition.

Tanka. (2004). BASIN SCALE RAINFALL - RUNOFF MODELING FOR FLOOD FORECASTS. Geoinformatics Centre, Asian Institute of Technology, Thailand, 10.

Technical Reference manual Hydrologic modeling system HEC_HMS. (2000).

US Army Corps of Engineers. (2010). HEC-RAS Version 4.1 Hydraulic Reference Manual. CA 95616: CDP-69USA.

USACE. (2000). Technical Reference manual Hydrologic modeling system HEC_HMS. US.

USACE. (2005). USA: U.S.Army Corps of Engineers.

Ven Te Chow, M. R. (1998). L.W Applied Hydrology . New York.

WMO. (2008). Guide to Hydrological practices.Volume I:Hydrology-From measurement to Hydrological information. Geneva: World meteorological organization.

ANNEXES

Appendix-A: Typical Hydrologic Soils Groups for Ethiopia

	Soil Types	Hydrologic Soil Group
Ao	Orthic Acrisols	B
Bc	Chromic Cambisols	B
Bd	Dystric Cambisols	B
Be	Eutric Cambisols	B
Bh	Humic Cambisols	C
Bk	Calcic Cambisols	B
Bv	Vertic Cambisols	B
Ck	Calcic Chernozems	B
E	Rendzinas	D
Hh	Haplic Phaeozems	C
Hl	Luvic Phaeozems	C
I	Lithosols	D
Jc	Calcaric Fluvisols	B
Je	Eutric Fluvisols	B
Lc	Chromic Luvisols	B
Lo	Orthic Luvisols	B
Lv	Vertic Luvisols	C
Nd	Dystric Nitosols	B
Ne	Eutric Nitosols	B
Od	Dystric Histosols	D
Oe	Eutric Histosols	D
Qc	Cambric Arenosols	A
Rc	Calcaric Regosols	A
Re	Eutric Regosols	A
Th	Humic Andosols	B
Tm	Mollic Andosols	B
Tv	Vitric Andosols	B
Vc	Chromic Vertisols	D
Vp	Pellic Vertisols	D
Xh	Haplic Xerosols	B
Xk	Caloic Xerosols	B
Xl	Luvic Xerosols	C
Yy	Gypsic Yermosols	B
Zg	Gleyic Solonchaks	D
Zo	Orthic Solonchaks	B

(Source: Ministry of Agriculture)

Appendix B: Values for the Computation of the Roughness Coefficient, n (CHOW, 1988)

	Channel Conditions		Values
Material Involved	Earth	n_0	0.02
	Rock cut		0.025
	Fine gravel		0.024
	Coarse gravel		0.028
Degree of irregularity	Smooth	n_1	0.000
	Minor		0.005
	Moderate		0.01
	Severe		0.02
Variations of channel cross section	Gradual	n_2	0.000
	Alternating occasionally		0.005
	Alternating frequently		0.01-0.015
Relative effect of obstruction	Negligible	n_3	0.00
	Minor		0.01-0.015
	Appreciable		0.02-0.030
	Severe		0.04-0.060
Vegetation	Low	n_4	0.005-0.01
	Medium		0.01-0.025
	High		0.025-0.05
	Very high		0.05-0.100
Degree of meandering	Minor	m_5	1.000
	Appreciable		1.150
	Sever		1.300

Appendix C: Homogeneity Test Analysis

Table C1 Station Shebe

Year	Jan	Feb	march	April	May	June	Jul	Aug	Sep	Oct	Nev	Dec	
1996	**69.1**	**49.7**	**243.3**	**301.2**	**184**	**137**	**190.5**	**152.7**	**196.5**	**42**	**71.1**	**43.3**	
1997	69.1	49.7	243.3	301.2	184	137	190.5	152.7	196.5	42	71.1	43.3	
1998	69.1	49.7	243.3	301.2	184	137	190.5	152.7	196.5	42	71.1	43.3	
1999	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2000	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2001	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2002	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2003	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2004	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2005	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2006	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2007	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2008	69.1	49.7	243.3	301.2	184	137	190.5	152.7	196.5	42	71.1	43.3	
2009	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2010	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2011	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2012	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2013	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2014	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	
2015	20.2	3.4	73.4	144.6	255	194	188.6	248.6	102	131.7	21.8	16.7	

sum	600	253	2148	3518	4814	3654	3780	4588	2418	2275	633	440.4	29121.2
mean	30	12.7	107.4	175.9	241	183	189	229.4	120.9	113.8	31.7	22.02	2426.8
pi	1.24	0.52	4.42	7.25	9.92	7.53	7.79	9.45	4.98	4.69	1.30	0.91	Shebe

Table C2: Station Bele

Year	Jan	Feb	March	April	May	June	Jul	Aug	Sep	Oct	Nev	Dec	
1996	**48.1**	**14.6**	**120.5**	**174.9**	**154**	**196**	**153.9**	**183.8**	**148.8**	**22.3**	**31.3**	**4.7**	
1997	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
1998	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
1999	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2000	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2001	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2002	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2003	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2004	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2005	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2006	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2007	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2008	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2009	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2010	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2011	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2012	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	

2013	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2014	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
2015	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	
sum	963	292	2411	3498	3078	3921	3077	3677	2976.8	446.6	626	93.1	25059.6
mean	48.1	14.6	120.5	174.9	154	196	153.9	183.8	148.8	22.3	31.3	4.7	2088.3
pi	2.3	0.7	5.772	8.375	7.37	9.39	7.368	8.803	7.1	1.1	1.5	0.2	bele

Table C3: Station Babu

Year	Jan	Feb	March	April	May	June	Jul	Aug	Sep	Oct	Nev	Dec	
1996	**51.7**	**29.6**	**189.2**	**153.9**	**226**	**185**	**279.9**	**140.3**	**149.1**	**71.4**	**101**	**29.4**	
1997	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
1998	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
1999	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2000	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2001	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2002	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2003	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2004	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2005	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2006	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2007	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2008	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2009	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	

2010	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2011	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2012	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2013	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2014	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
2015	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	
sum	1034	592	3784	3078	4524	3706	5598	2806	2982	1428	2022	588	32142
mean	51.7	29.6	189.2	153.9	226	185	279.9	140.3	149.1	71.4	101	29.4	2679
pi	1.93	1.11	7.064	5.746	8.45	6.92	10.45	5.238	5.567	2.666	3.77	1.098	babu

Table C4: Station Sawula

Year	Jan	Feb	march	April	May	June	Jul	Aug	Sep	Oct	Nev	Dec	
1996	**63.547**	**44.6**	**156**	**210**	**169.7**	**222**	**168**	**132**	**143**	**53.8**	**40**	**10.9**	
1997	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
1998	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
1999	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2000	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2001	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2002	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2003	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2004	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2005	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2006	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2007	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2008	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2009	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2010	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2011	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2012	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2013	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2014	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	
2015	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	

sum	1270.9	891	3112	4199	3393.9	4445	3359	2642	2858	1075	795	218	28261
mean	63.547	44.6	156	210	169.7	222	168	132	143	53.8	40	10.9	2355.1
pi	2.6983	1.89	6.61	8.916	7.2056	9.44	7.13	5.61	6.07	2.28	1.7	0.46	Sawula

Appendix D: Areal Meteorological Data Computation

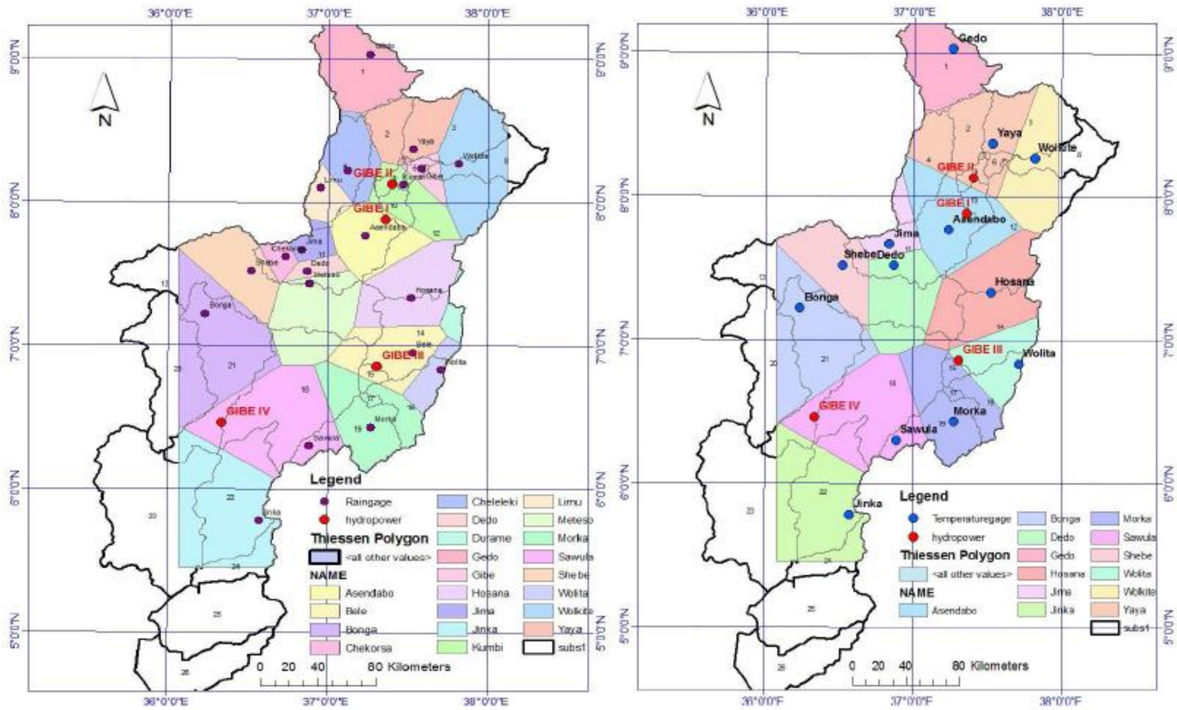


Figure D: Theisen polygon developed for Omo Gibe sub watersheds of precipitation (left panel) and temperature (right panel)

Appendix E: HEC- HMS Outputs

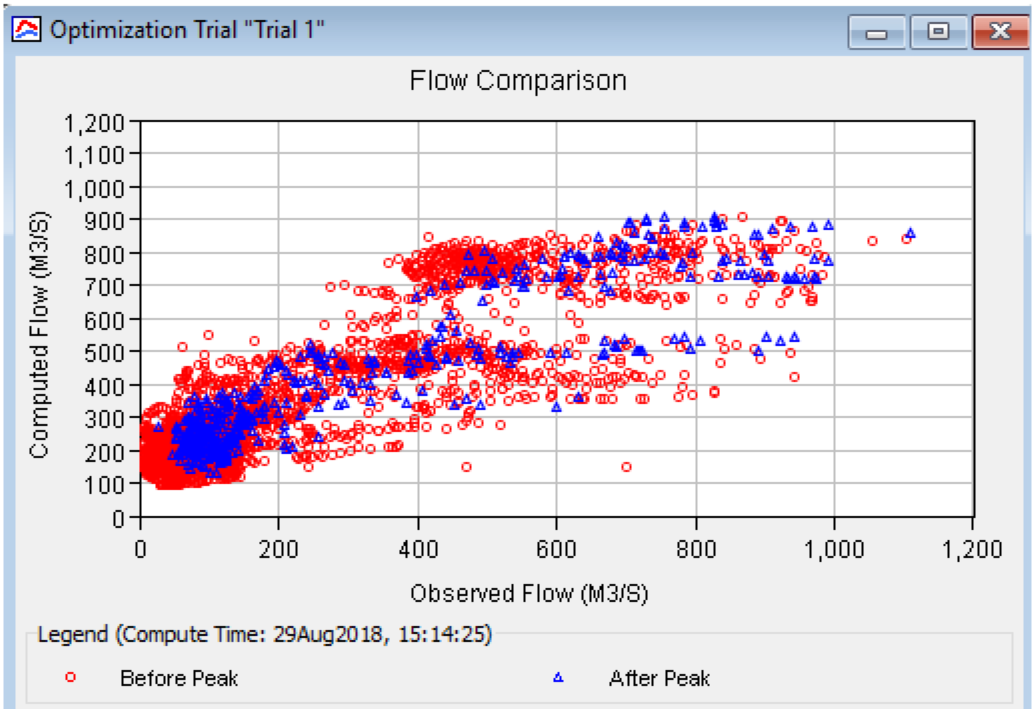


Figure E1. Observed flow versus Computed flow

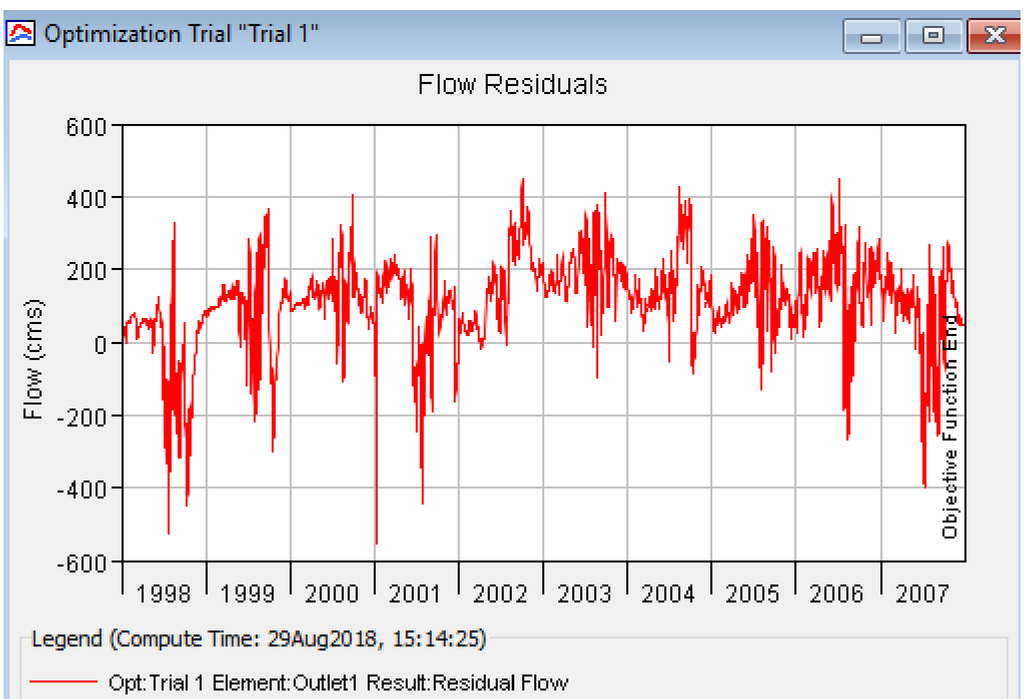


Figure E2: Flow Residuals

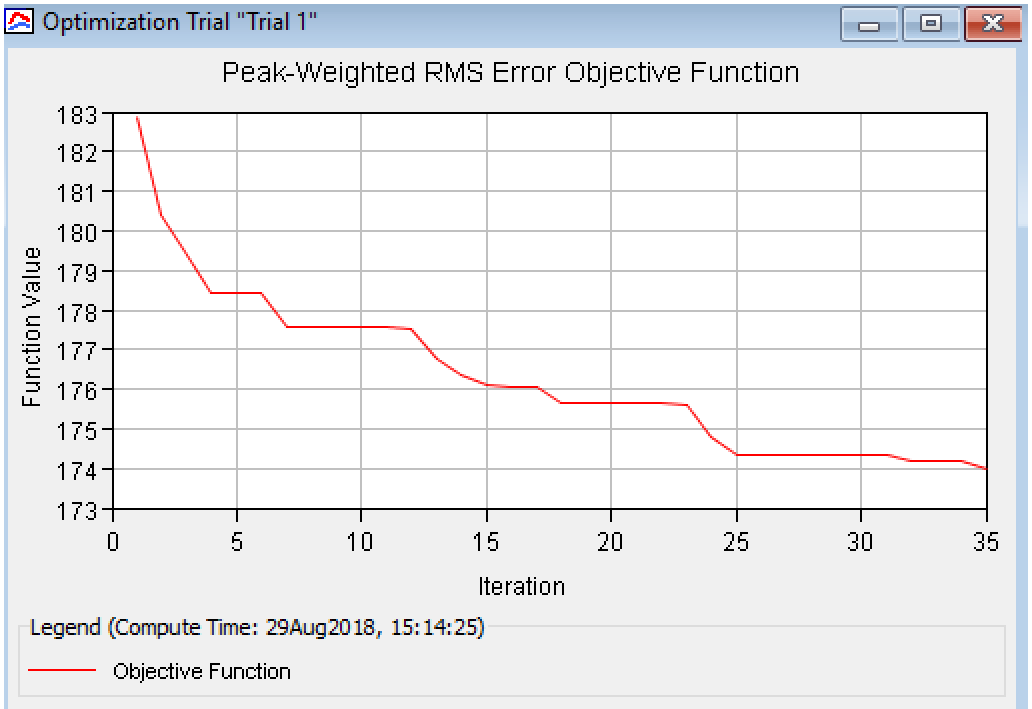


Figure E3: Peak-Weighted RMS Error Objective Function

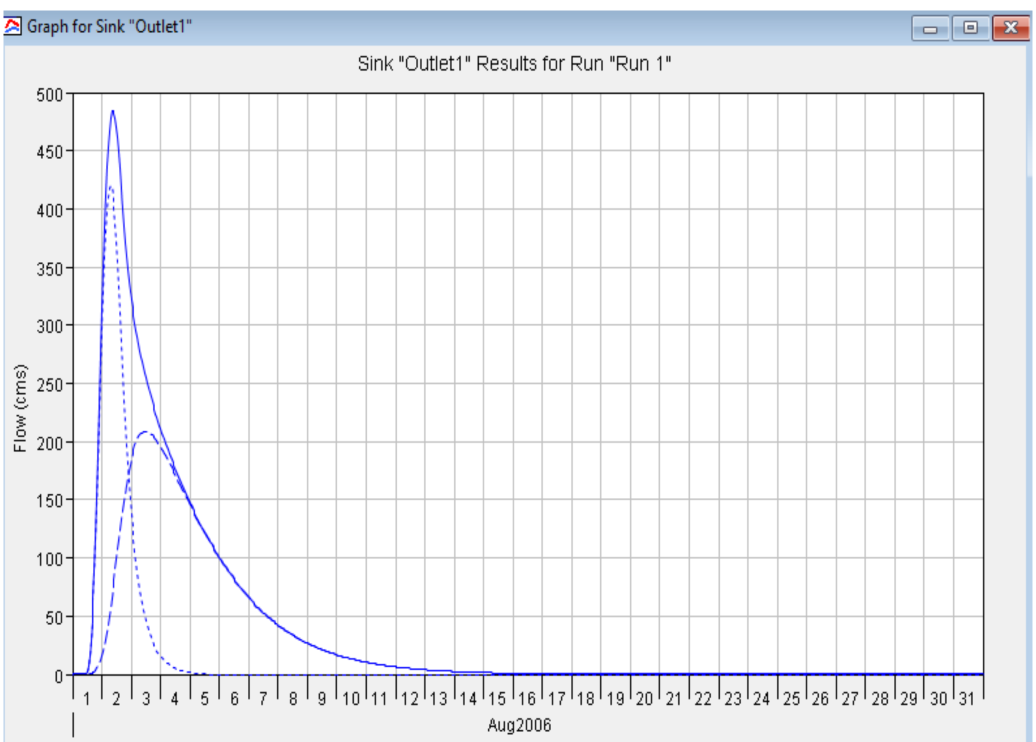


Figure E4: 2-year flow hydrograph of Lower Omo Gibe River

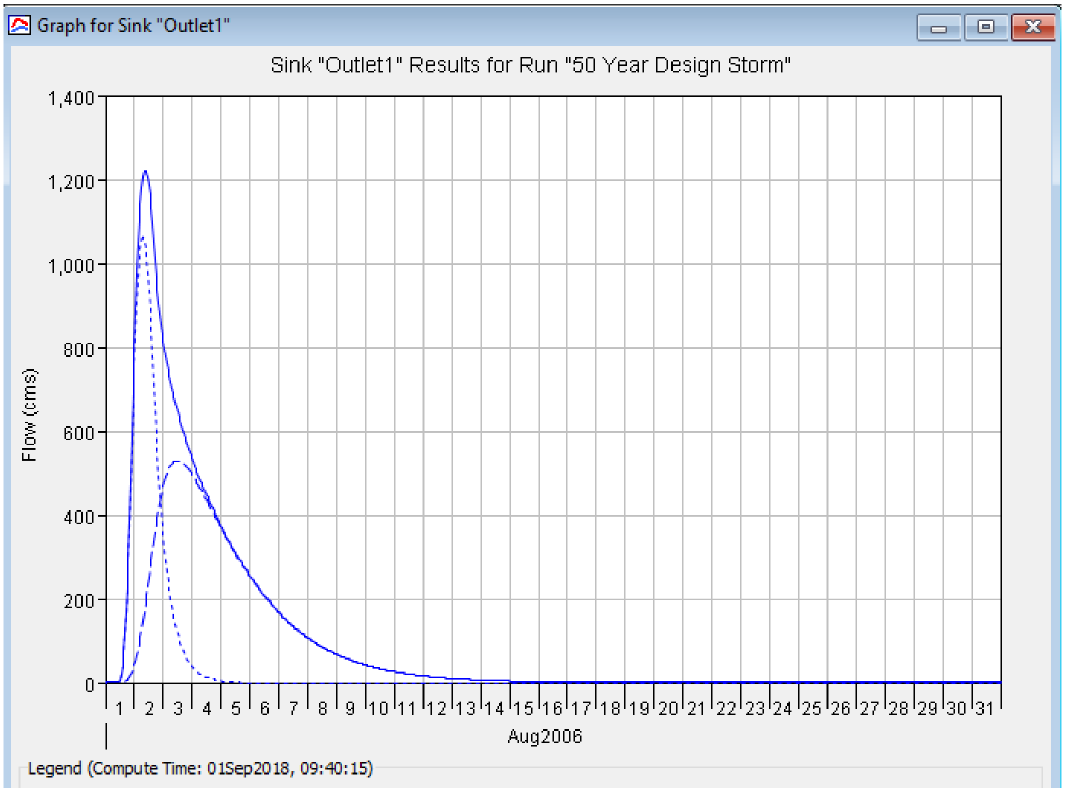


Fig E5: 50-year flow hydrograph of Lower Omo-Gibe River

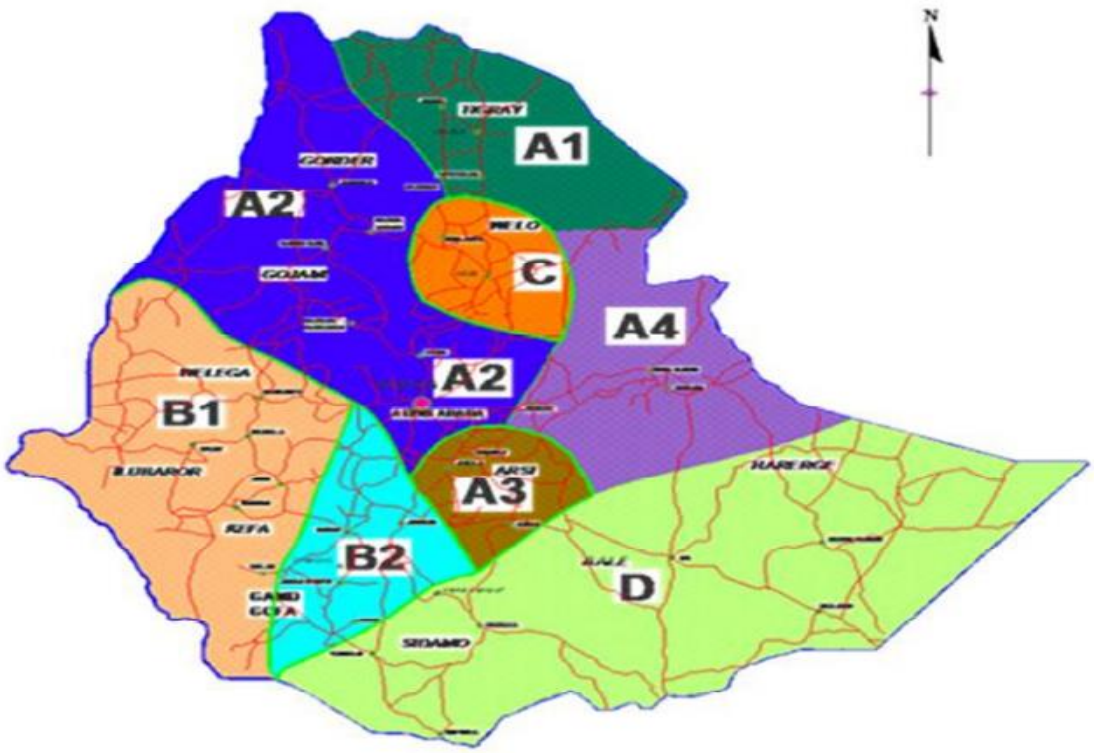
Appendix F: Estimated Areal Precipitation and Evapo-transpiration

Table F Mean Monthly Areal Rainfall [mm]

Year	Jan	Feb	March	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1996	55.41	31.57	155.23	185.75	189.97	210.86	201.36	158.05	153.61	48.12	55.41	15.07
1997	43.08	1.41	46.33	181.07	187.03	149.97	139.31	163.66	105.17	274.63	207.05	68.91
1998	78.10	46.32	73.59	135.68	181.66	183.62	201.37	169.06	142.11	218.87	47.99	2.90
1999	16.29	0.84	61.15	106.46	145.44	137.54	158.53	144.84	85.69	200.38	13.31	18.56
2000	0.64	0.66	29.41	128.31	197.04	187.02	145.74	187.17	166.39	217.54	46.49	31.01
2001	18.18	26.90	114.38	133.80	204.65	176.82	211.93	212.70	171.06	138.90	43.10	21.37
2002	67.10	7.57	146.13	93.75	177.49	115.11	162.41	189.06	127.68	62.25	11.07	128.68
2003	56.22	33.92	72.86	157.02	110.07	191.83	174.73	207.77	138.66	71.82	51.27	56.12
2004	54.39	26.86	41.25	189.08	107.47	131.03	162.39	184.13	140.05	92.37	80.92	51.79
2005	36.33	15.43	128.57	147.45	320.53	128.80	163.32	173.45	176.86	108.92	92.16	2.11
2006	18.80	49.09	163.32	150.43	194.46	146.62	196.24	246.48	150.45	169.21	74.35	133.42
2007	56.56	52.14	71.13	133.80	221.96	232.46	187.09	211.75	260.11	52.21	22.09	1.35
2008	10.34	14.17	31.79	95.26	184.18	186.12	196.30	226.46	193.83	160.38	92.63	3.74
2009	32.50	13.88	54.61	145.14	101.67	94.01	116.91	139.48	167.25	153.21	61.85	113.18

2010	18.60	66.59	133.38	165.52	221.93	153.31	160.05	215.25	202.61	57.94	41.58	38.76
2011	4.19	9.69	52.88	82.15	204.12	122.38	226.84	314.44	220.09	49.04	114.55	11.08
2012	0.31	1.57	23.03	188.42	123.31	156.78	223.10	218.80	251.88	38.66	97.53	48.29
2013	29.50	15.00	106.17	256.43	111.43	132.52	194.02	258.04	256.70	94.37	72.75	2.41
2014	12.06	17.18	126.05	109.18	234.29	124.38	161.87	284.98	177.22	232.94	83.77	7.69
2015	0.23	10.54	75.75	172.44	174.69	174.56	158.63	180.21	184.20	117.14	130.66	120.82

Appendix G: Drainage Design Manual Nov.13

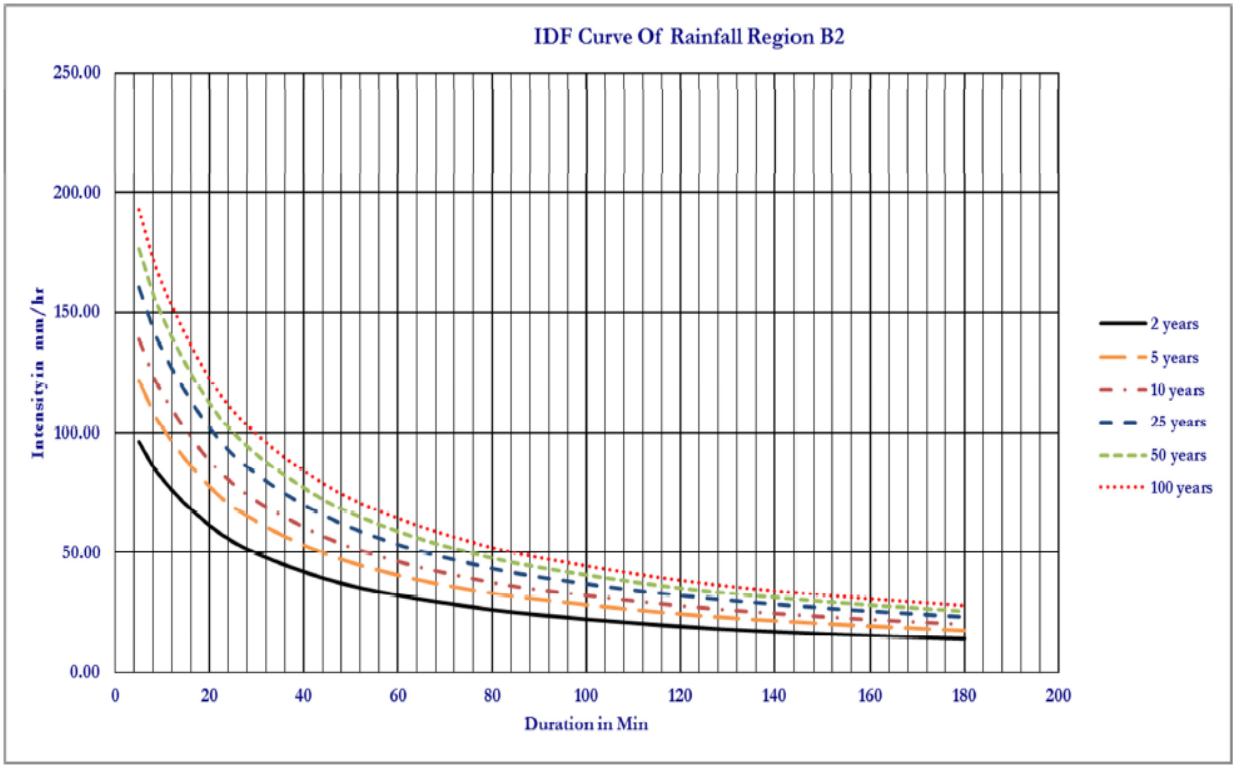


FigureG1: Rainfall Region. Source: MWIE

Table 5-19: 24hr Rainfall Depth Vs Frequency

Return Period Years	24 hr Rainfall Depth (mm) vs Frequency (yr)							
	2	5	10	25	50	100	200	500
RR-A1	50.30	66.02	76.28	89.13	98.63	108.06	117.48	130.00
RR-A2	51.92	65.52	74.45	85.70	94.07	102.45	110.91	122.27
RR-A3	47.54	59.61	67.66	77.92	85.62	93.34	101.13	111.58
RR-A4	50.39	63.83	72.28	82.55	89.97	97.20	104.32	113.63
RR-B1	58.87	71.26	79.29	89.35	96.84	104.37	112.02	122.41
RR-B2	55.26	69.95	79.68	92.03	101.29	110.61	120.07	132.87
RR-C	56.52	71.04	80.54	92.52	101.48	110.50	119.66	132.06
RR-D	56.23	76.84	90.37	107.46	120.23	133.05	146.00	163.44

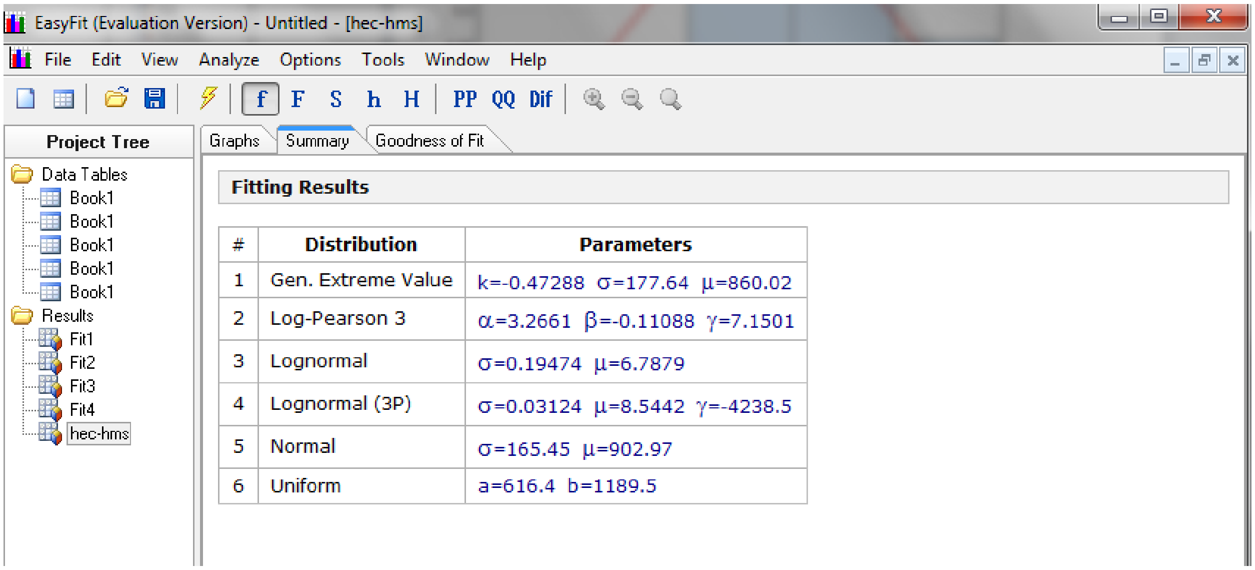
Note: RR- Rainfall Region



FigureG2, IDF Curve for Rainfall Region B2

Appendix H: Easy Fit Software Out put

Table H1: Summary Statistics of the Yearly Discharge data for the Outlet during the year (1996-2015)



Fitting Results

#	Distribution	Parameters
1	Gen. Extreme Value	k=-0.47288 σ=177.64 μ=860.02
2	Log-Pearson 3	α=3.2661 β=-0.11088 γ=7.1501
3	Lognormal	σ=0.19474 μ=6.7879
4	Lognormal (3P)	σ=0.03124 μ=8.5442 γ=-4238.5
5	Normal	σ=165.45 μ=902.97
6	Uniform	a=616.4 b=1189.5

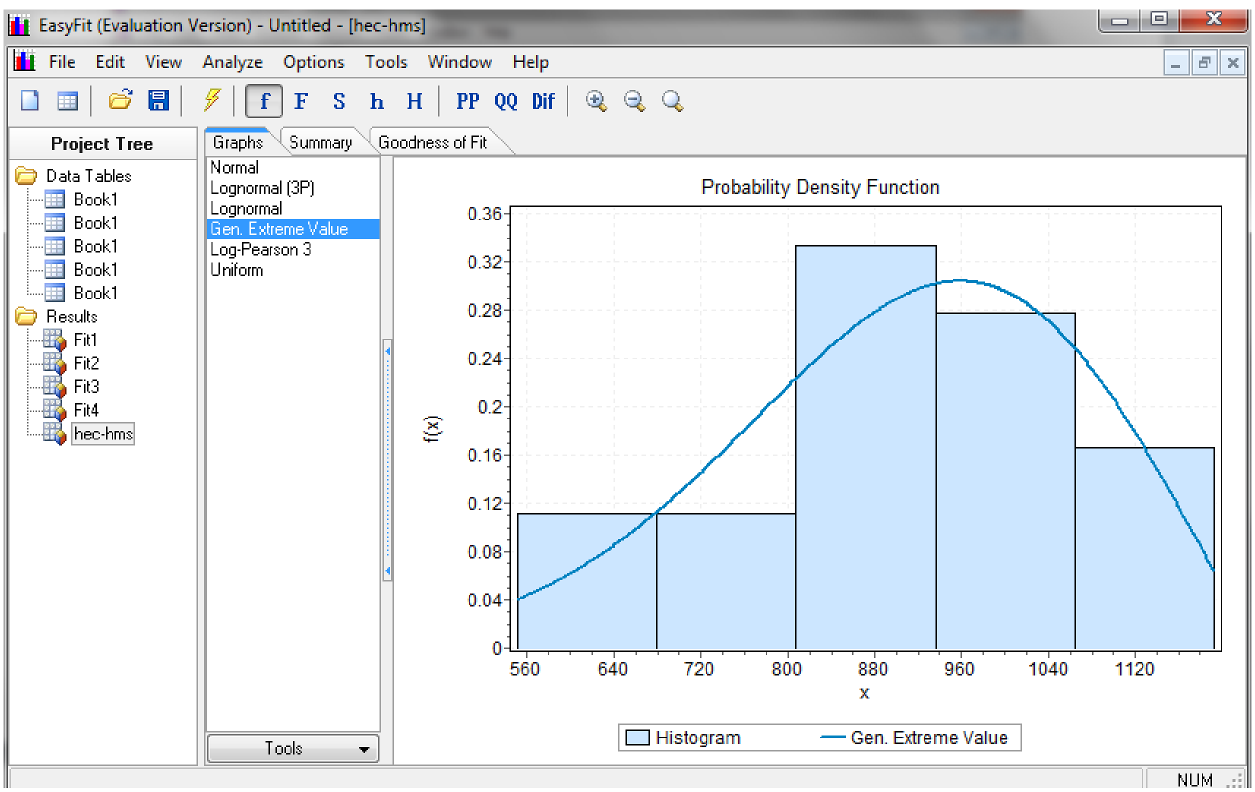


Figure H1:PDF for general Extreme Value distribution

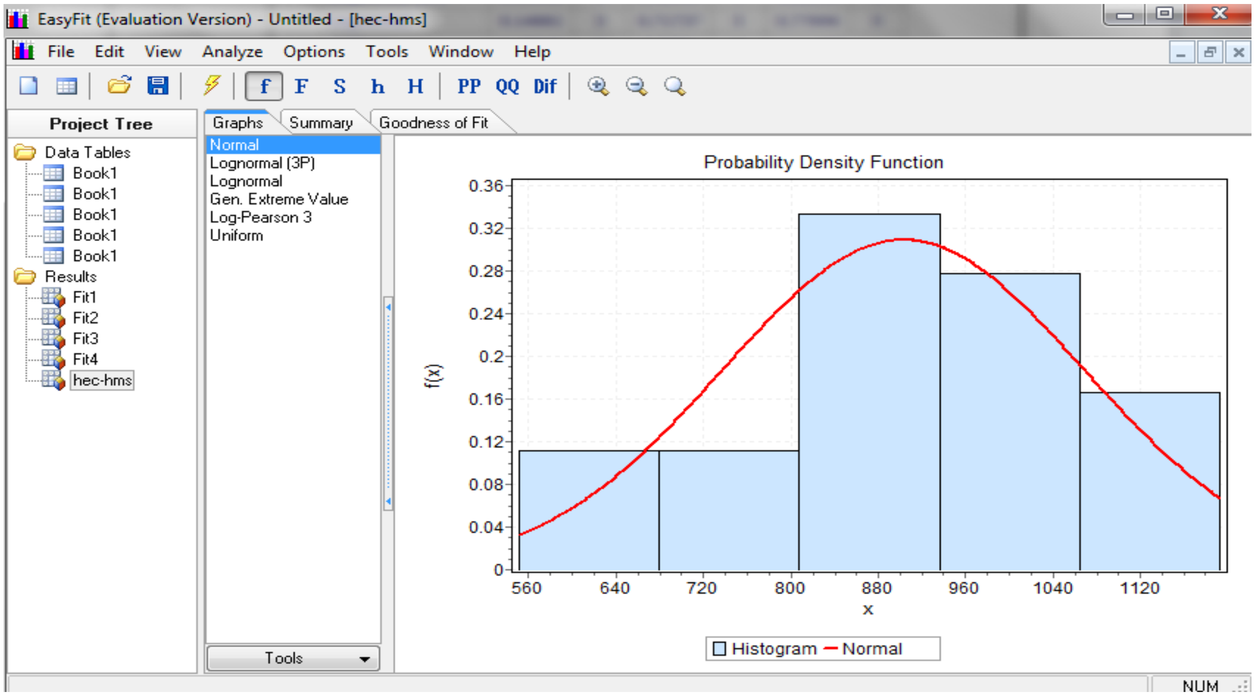


Figure H2: Probability density Function for Normal Distribution

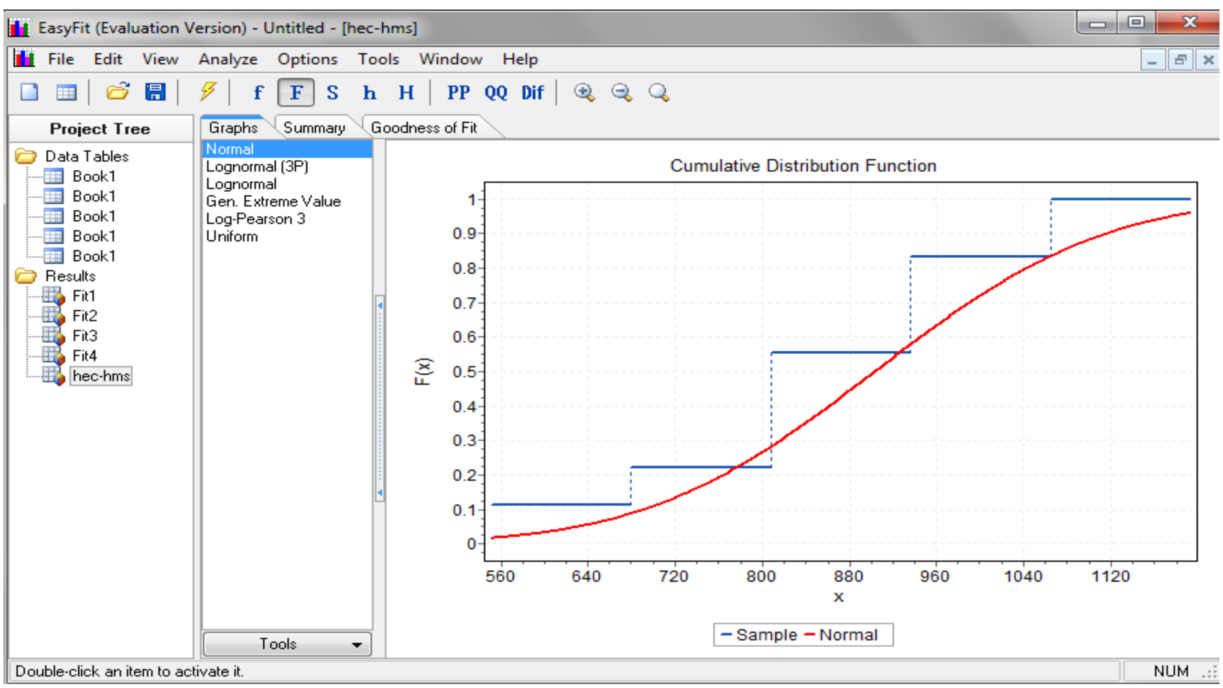


Figure H3: Cumulative Distribution Function for normal distribution

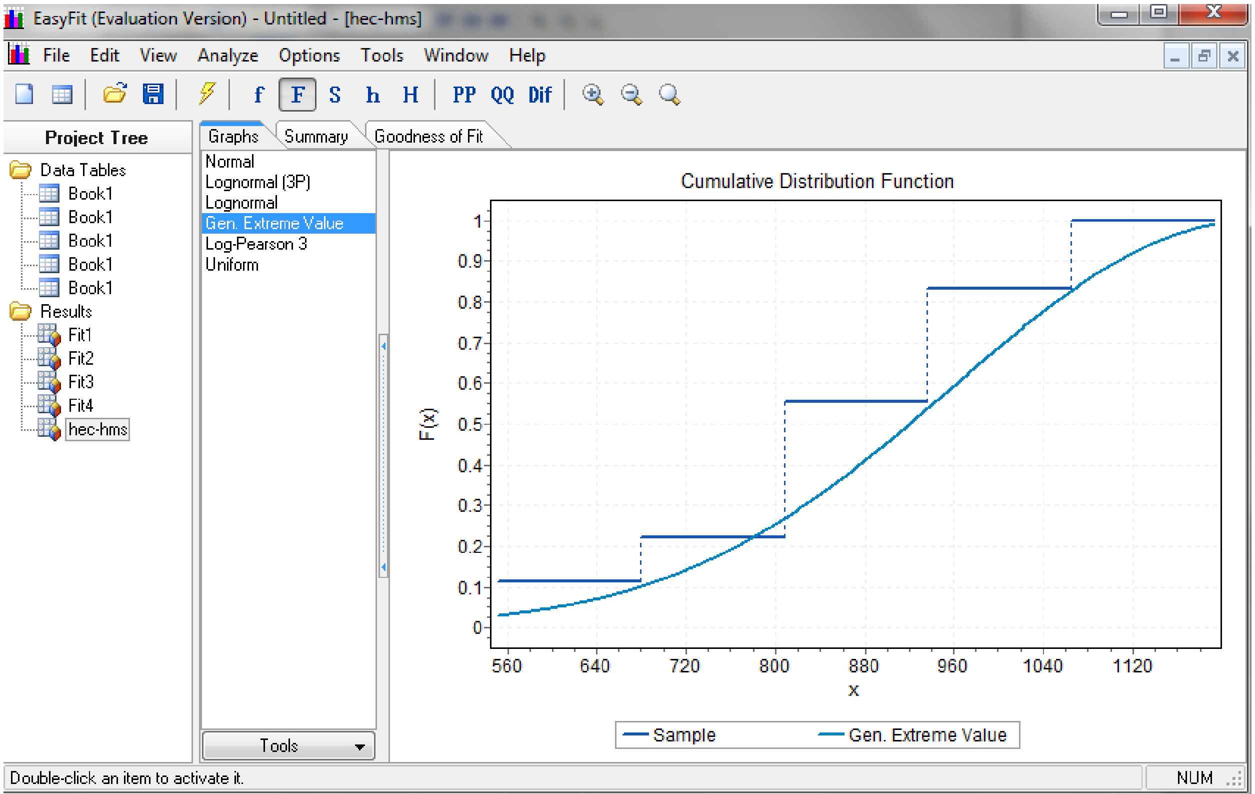
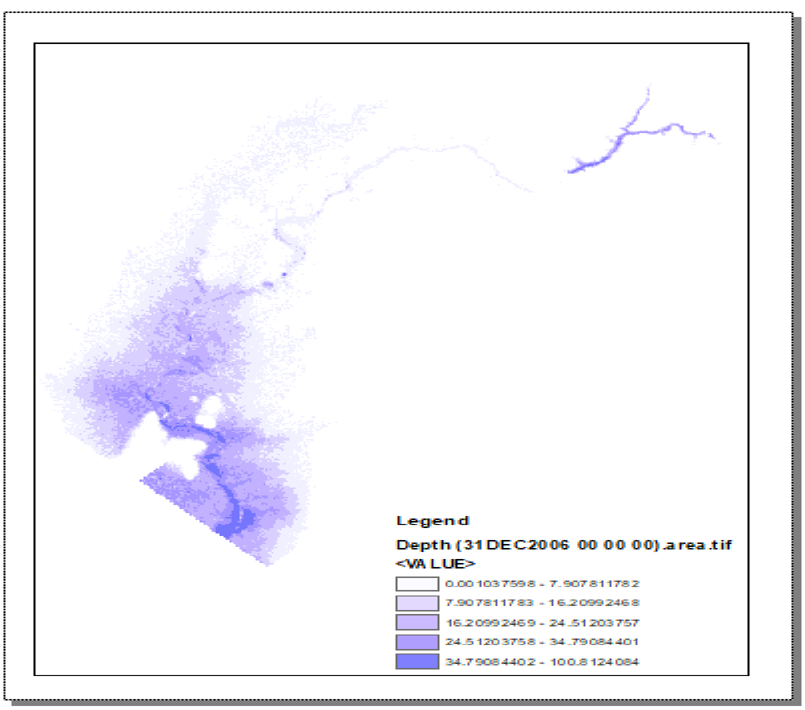
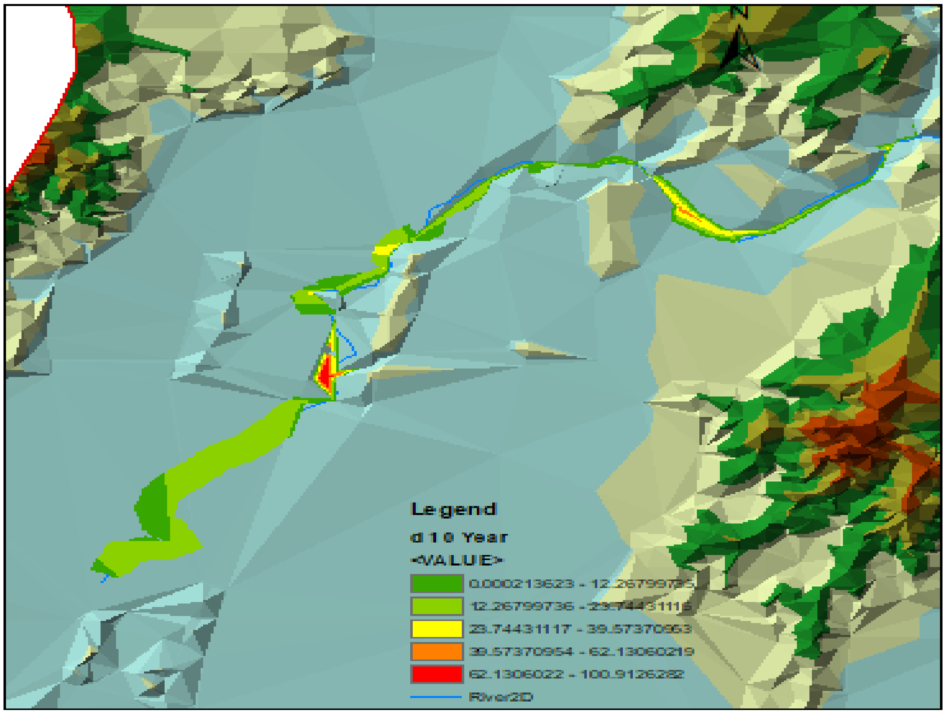


Figure H4: Cumulative Distribution Function for normal distribution

Appendix I: 2D Flood Mapping



FigureI1: Exported 2D Maps from HEC-RAS to Arc-Gis by using unsteady flow data



FigureI2 :10-year flood map and depth for the study area for mixed flow

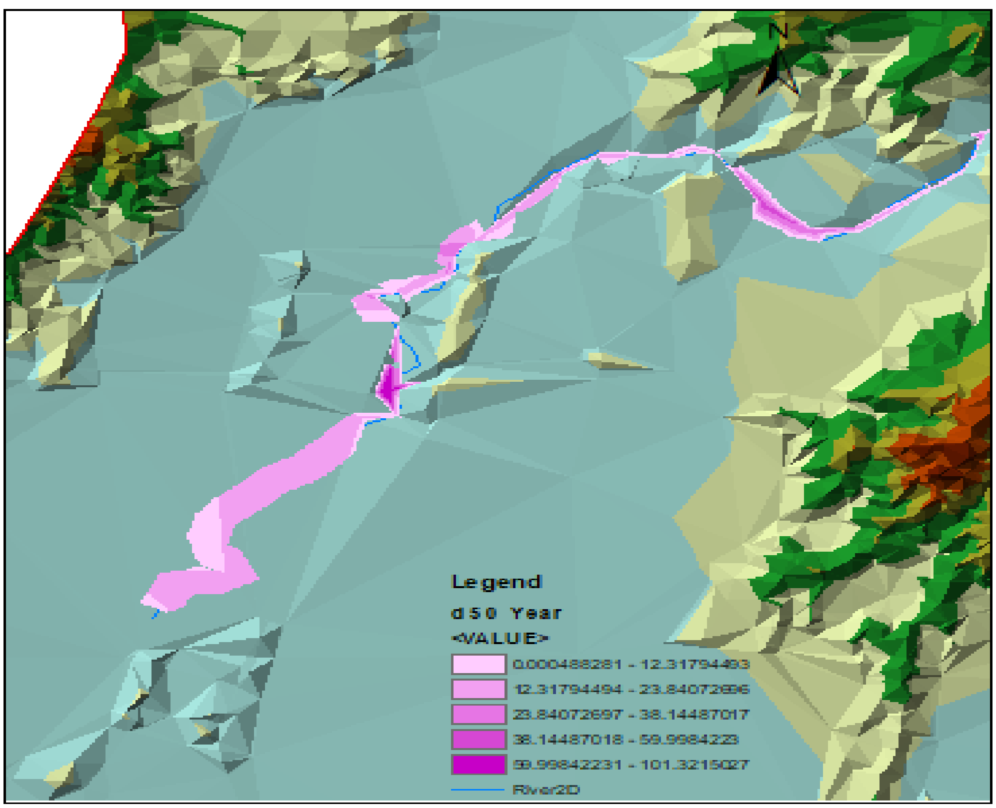


Figure I3:50-year flood map and depth for the study area

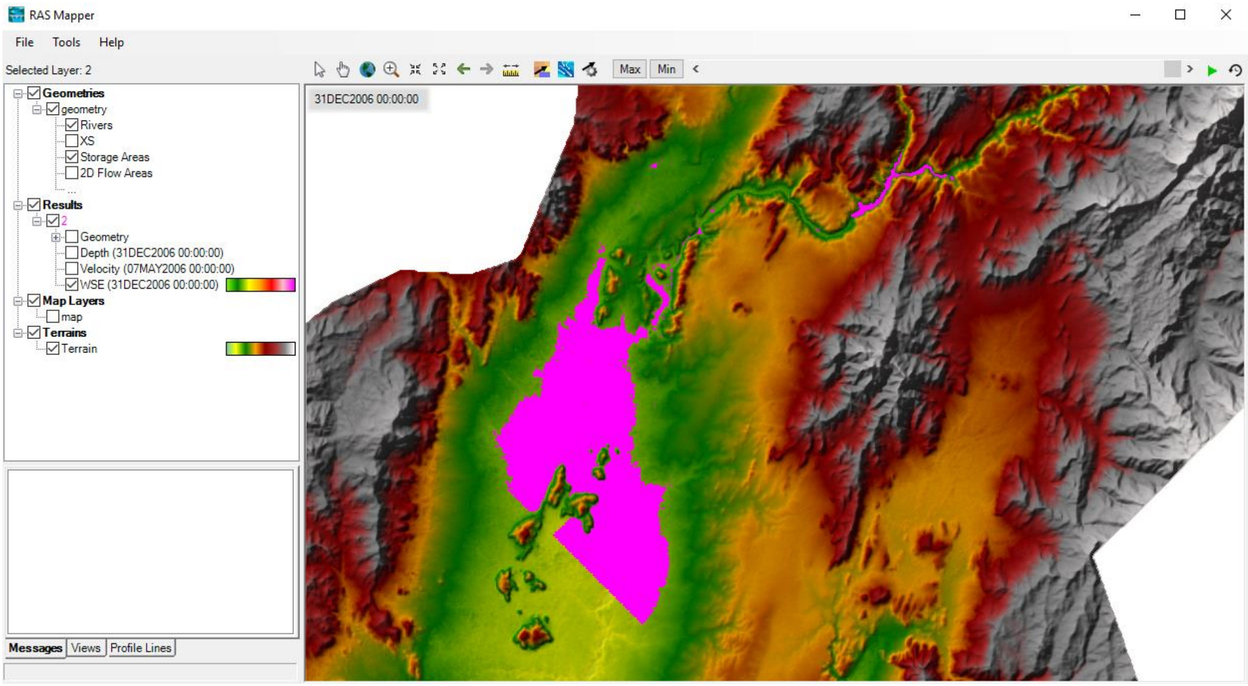


Figure I 2: Computation flood water surface profile in RAS mapper at the Omo Kuraz district

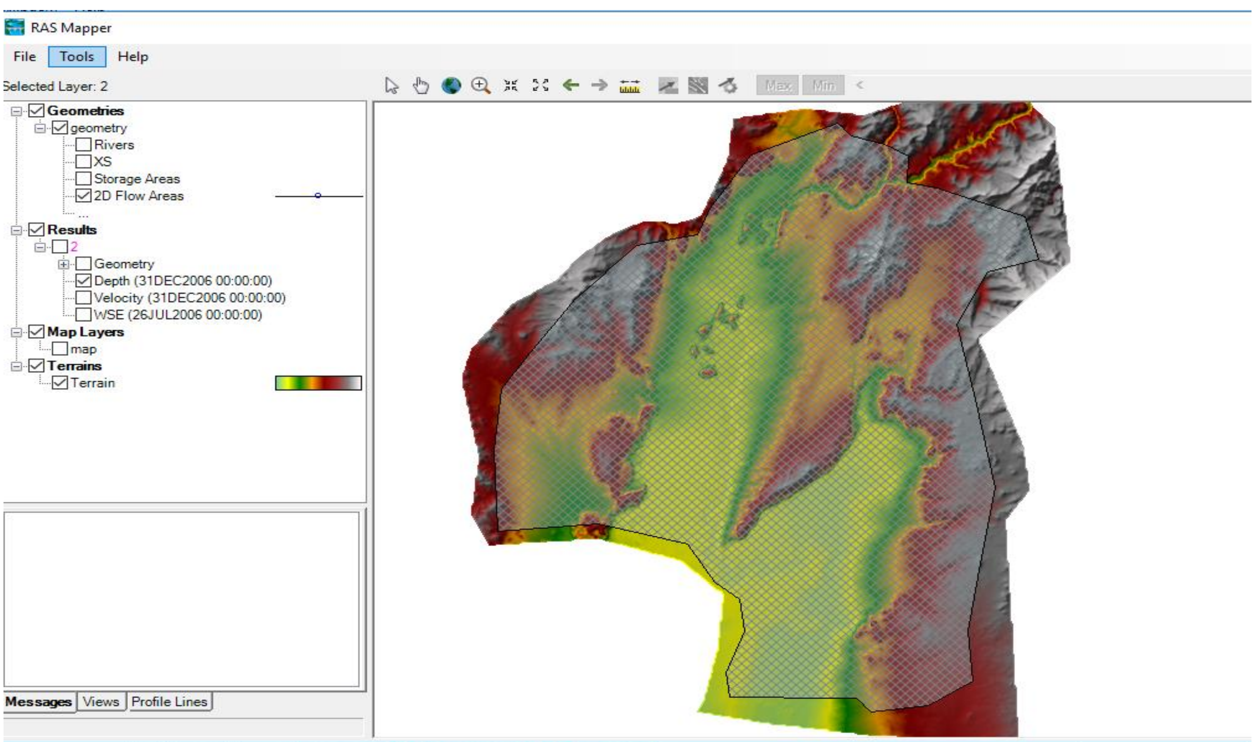


Figure I3: Flood area grid computation in RAS mapper at the Omo Kuraz district

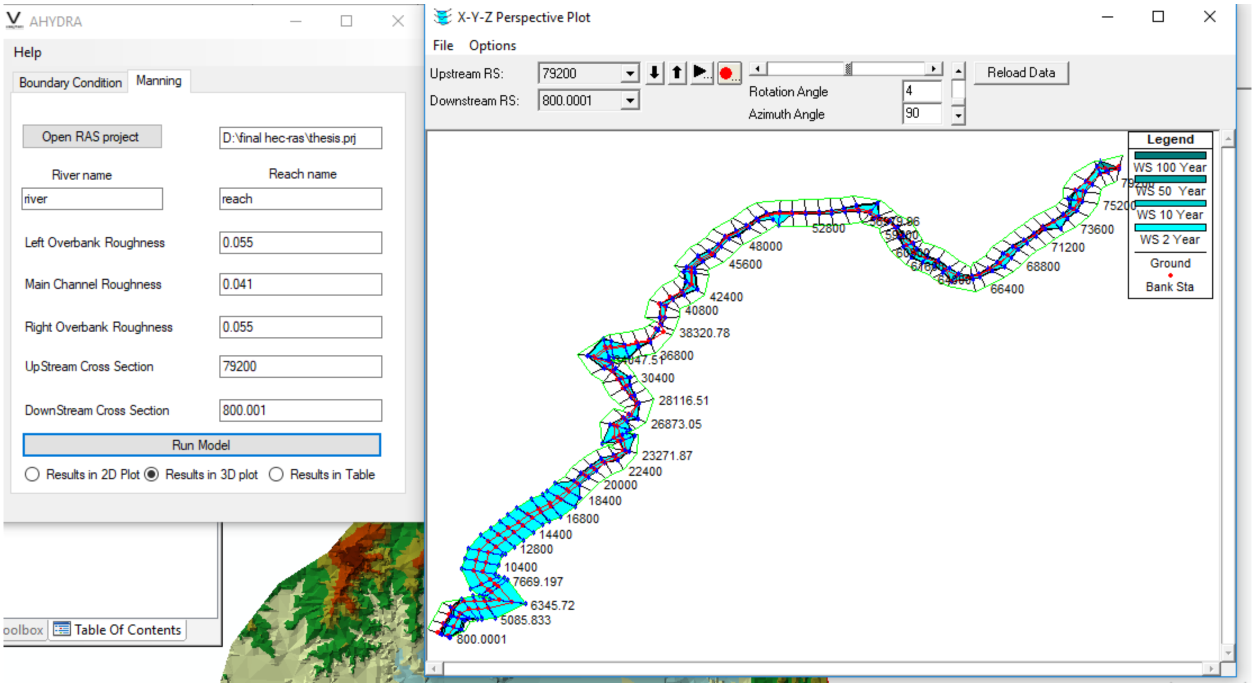


Figure I4 X-Y-Z perspective plot with AHYDRA